MAY 1 2 1928

Devoted to Civil Engineering and Contracting. McGraw-Hill Publishing Company, Inc.

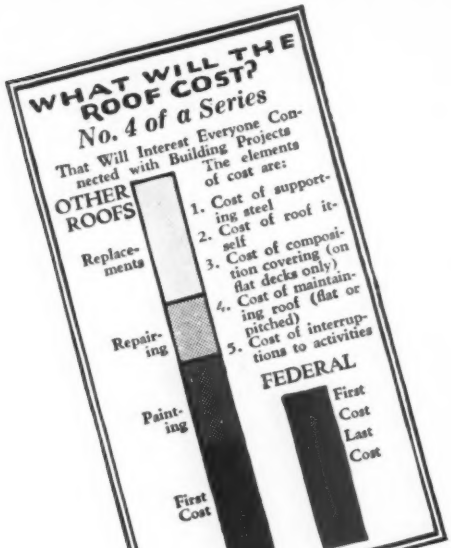
May 10, 1928

Engineering News-Record

Transferring Excavated Material From Cut to Fill Across Double-Track Railway on Highway Work Near Wilkes-Barre, Pa.

St. Francis Dam—Facts and Findings Concerning the Failure. A Summary by N. A. Bowers. Also Abstracts of Reports by Four Boards of Engineers and Geologists

Maintenance? There is none on a Federal Roof!


WHAT WILL THE ROOF COST?
No. 4 of a Series
That Will Interest Everyone Connected with Building Projects
The elements of cost are:
1. Cost of supporting steel
2. Cost of roof itself
3. Cost of composition covering (on flat decks only)
4. Cost of maintaining roof (flat or pitched)
5. Cost of interruptions to activities
OTHER ROOFS
Replacements
Repairing
Painting
First Cost
FEDERAL
First Cost
Last Cost

Of all parts of a building, the roof is certainly not one to "buy for the present and let the future take care of itself!"

No part of the building has to stand the punishment that a roof does. It is not surprising, therefore, that buyers *expect* to spend money on painting, repairing, and eventually replacing the ordinary roof. Maintenance on common roof materials cannot be ignored—it must come and must be paid for.

But why spend this money uselessly? A Federal Roof is paid for but once. It is concrete, immune to every effect of time and weather, to smoke, steam or gases—to fire. There are no costly interruptions to production or loss of business. All maintenance expense is automatically eliminated—a saving more than sufficient to roof another building!

Our engineers have these and other facts for your analysis. Ask for details—no obligation whatever.

Made, Laid and Guaranteed by
FEDERAL CEMENT TILE CO.
608 S. Dearborn Street • Chicago
FOR OVER A QUARTER CENTURY

Wolverine Tube Co., Detroit. Federal Channel Slabs form a permanent, fireproof, roofdeck for flat or sloping surfaces, to take the usual composition covering.

No Building Can Be Firesafe Without a Fireproof Roof

Commonwealth Steel Foundry, Granite City, Ill., covered with Federal Red Interlocking Tile, which saves cost of composition covering. Note: Though walls are steel sheets, roof is permanent.

FEDERAL CEMENT TILE

A Consolidation of Engineering News, Engineering Record and The Contractor

ENGINEERING NEWS-RECORD

DEVOTED TO CIVIL ENGINEERING AND CONTRACTING

McGraw-Hill Publishing Co., Inc.
JAMES H. McGRAW, *President*
E. J. MEHREN, *Vice-President*

WILLARD CHEVALIER
General Manager
F. E. SCHMITT
Managing Editor

Volume 100 NEW YORK, MAY 10, 1928 Number 19

Plain Facts

WITH the tenacity of all vogues, the subject of unemployment that was a matter of vogue in public-affairs discussions a month or two ago still maintains some hold on the popular attention. Plain facts show even more clearly, however, that it is unjustified talk. Thus statistics of the National Industrial Conference Board as quoted recently by its president, Magnus W. Alexander, indicate that manufacturing employment was higher in February than at any time since last June. The statement gives a solid basis for cogitation on two phases of employment. First, are there abrupt variations in consumption, through which up or down swings of production-balance and therefore of employment would be caused? On this point all facts brought out contradict the existence of such variations in normal periods like the present. Second, do the steadily progressing substitution of machine for muscle and the increasing rate of production per worker constitute a pressure on the labor market? Here again the answer is negative. The facts, therefore, give no support to pessimistic utterances concerning employment. Since, also, construction contracts of the year so far are in excess of those for the same months of last year and are at peak level, it is apparent that industry's pace is strong.

Reservoir Deposits

DEPOSITION of suspended matter in reservoirs is one of the long-standing and yet troublesome elements in hydraulic engineering. In this country it concerns mainly the West, for there the streams carry heavy loads of silt and in some instances of sand and gravel. Silt deposition is the more common problem, so far without solution other than provision of enough storage capacity to make the depletion of reservoir volume unimportant. Gravel movement is a more special and localized problem, which not only may affect storage volume but involve the further danger of interference with gates and other operating elements of the dam. An attempt to deal with the expected difficulties has been made in a dam on a California stream which carries old mining débris in flood time. Sluices through the dam are to be so operated as to carry flood débris through before storage begins. The experiment is interesting, especially because it will provide information as to the movement and manner of deposition of the material. Whether the gravel will be carried down to and through the gates or will bank up in the upper part of the reservoir, and whether the handling of the gates will be affected by the deposition even under the operating scheme contemplated, should form the theme of valuable reports after some seasons of operation—unless a dam now being built above this one interrupts the experiment. But it is to be borne in mind that the adopted method depends on abandoning storage of the stream flood waters, which in many cases are the very waters to be saved. To this extent it does not fall in the class of generally available remedies.

Muddy Seepage

THE St. Francis disaster has put the matter of seepage under dams in a new light—that is, new to those unaccustomed to dealing with rock having a relatively high content of soluble minerals. One per cent of soluble matter may be quite serious if this percentage is distributed throughout the mass in veins, as is often the case. There has been a widespread tendency to look for muddy seepage as the danger sign, and in the absence of turbidity, or of large quantity of flow, to feel no anxiety about leakage through or under dams. The St. Francis failure stresses the importance of chemical analyses of clear seepage. Such analyses should be made when storage is begun and often enough thereafter to keep a reasonable check on possible changes in conditions. If such an analysis had been made at the St. Francis dam a week before the failure, according to the data presented elsewhere in this issue, simple arithmetic would have shown the investigators that elements were being brought from under the dam by clear seepage water at such a rate that the very small flow of 2 miner's inches (0.04 sec.-ft.) could have removed 97 tons, or 1,343 cu.ft., of solid material in the fifteen months that the reservoir had been in service. This experience is a warning that clear seepage is not necessarily harmless; many minerals are soluble, and only a chemical test will show what clear water really contains.

Water to the Fore

IT IS worth while to consider how large a part of recent happenings, discussion and negotiation has been concerned with water. More than once in recent months the current of development as reflected in these columns as well as in the pages of the daily press has centered on water questions. Unless this is only a momentary condition, it means that water is entering far more deeply and intimately into our manner of living than ever before. Power, floods, irrigation, purification, storage for supply or equalization, control for shipping, bridge problems, not only engage much of the activities of civil engineers but form the basis of many important public questions. The dominant public issues for some years have involved water. Legislators, negotiators and courts have been concerned with the Colorado problem, Mississippi flood control, reclamation, water-power administration, the Muscle Shoals plant and the St. Lawrence and Great Lakes questions. Water adjudication with Canada on the north and Mexico on the south is a continuing problem. In part, these facts may be connected with the diversity of rights and responsibilities in the field of water—the

heterogeneous legal status of state, federal and private rights. But in the last analysis the decisive fact is the great and growing importance of water to modern man. On every present prospect, the future will see this importance increased still more.

Short-Lived Sewage-Works

TEN years is the average life of a plant for treating sewage if reliance can be put on the experience of Madison, Wis., which is just abandoning its third plant and putting its fourth in use. Madison has been unfortunate with its sewage-works. It and the promoter were justly punished for the immediate failure of the first plant, built along the lines of a British proprietary process, untried here and not extensively used elsewhere. The short lives of the two succeeding plants are attributed locally to an unforeseen rapid growth of Madison. Whichever or whatever the cause, the outlook for sewage-works construction would be unfavorable if a longer life than ten years could not be expected. But can it? Unfortunately, there are no data available to show whether the experience of Madison is exceptional. Offhand, one would say that it is; but many cities have either rebuilt their sewage-works a few years after they were put in use or should have done so if works of any kind were warranted in the first instance. That this is more true of sewage-works than of some other municipal improvements should not be assumed as a certainty. The case seems to be far worse for garbage-disposal plants. Probably it is considerably better for water-treatment plants and also for water-pumping plants, for the recent sweeping changes in the latter followed long years of service for the displaced pumps. More and better data on the useful life of all classes of municipal improvements would be welcome. We fear they would not justify the short-livedness of the Madison sewage-works, but they might give some crumbs of comfort to those responsible for what has happened there. Fortunately, the newest works at Madison have been planned with an eye to future expansion.

Conservatism and Vision

IN THE record of a discussion of vehicle tunnel operation, held before the American Society of Civil Engineers less than ten years ago, there may be found an interesting contrast between engineering foresight and the subsequent facts. A speaker who had unusually complete knowledge of local problems estimated the capacity of a pair of two-line tunnels, carrying traffic at 3 and 9 miles per hour respectively for the slow and the fast line, at some 40,000 vehicles in twelve hours. This, he added, "is five times the traffic required at their completion and would be ample for the traffic in 1942." But last year, when the Holland tunnels were opened, the speed of traffic almost at once adjusted itself to about 20 and 35 miles per hour—as against the engineer's prediction of 3 and 9. The traffic volume began with a one-day figure of about 50,000 and soon settled to an average of about 20,000, against the 8,000 foreseen. Thus it appears that the judgment of eight years previous proved to be absurdly inadequate. In part, no doubt, this is because the forecast was intentionally conservative; the engineer notoriously tends this way, and by the very necessities of his work in planning improvements he must be so, since overoptimistic forecasts would jeopardize the soundness of large expenditures. But it is well worth considering whether conservatism is not sometimes out of place. In foreseeing the future it is possible to err as gravely through inadequate vision as through exaggerated expectation. Lack of vision is frequently charged to engineers as a fault, and it may often prove a fatal fault. The prime mission of the engineer is to foresee truly, to appraise future happenings neither too high nor too low, but with precision. Admitting the difficulty of maintaining sound conservatism in harmony with hopeful vision, we are bound to recognize that the engineer's training and career urge him so steadily toward conservatism that he is likely always to be in need of cultivating vision. Imagination is a faculty which the engineer needs as much as any other man; perhaps more.

Less Railway Competition

NO MORE paralleling railway lines will be built in the future if the reasoning of the Interstate Commerce Commission in the Piedmont & Northern case is followed in passing upon proposed railway construction. The Piedmont & Northern asked authority to extend its lines over routes which closely parallel the Southern Railway through the Piedmont section of the Carolinas. Except for the other railways, everyone in the region, including state public service commissioners, approved the building of the road. But the Southern Railway claimed that it serves the region adequately and has plenty of excess capacity for new business, now developing rapidly. From most points of view except that of the Southern Railway the case appeared to be one in which the new construction should have been approved if the United States is to hold to the policy of encouraging reasonable railway competition. The railway with which the enlarged P. & N. would compete is not a poor line dependent upon traffic from the Piedmont section for its existence. Moreover, it should not fear that it will lose its present traffic to the P. & N. if it can offer as good service and routing as the P. & N., which it claims it can. Competition, then, would be chiefly for the new traffic. If the P. & N. was willing to risk large expenditures for that purpose, the region should have been granted the benefit of the increased service.

The reasoning of the preceding paragraph purposely disregards one point which should in no way have been allowed to affect a decision as to whether or not the new lines should be built, but which, we believe, nevertheless has been considered because the commission saw fit to devote considerable space to it in summing up its conclusions. It has to do with the possibility of the Piedmont & Northern Railway being able to influence shippers to route traffic over its lines on account of its close connection with the utilities and industries of the region. The railway is controlled by the Duke estate. The same interests control the Southern Power Company, which not only supplies most of the power to local industries but also ships into the region large quantities of coal for its own consumption. Moreover, many of the railroad's directors are directors or men otherwise interested in local industries. Nothing developed at the commission's hearings to indicate that any of these relationships had been used to gain traffic for the P. & N., but the commission took cognizance of the fact in stating that it is reasonable to conclude that there will be a unity of interest between the railroad company and the other Duke properties.

Consideration of all these conditions in connection

with an application for authority to build new railway lines confused two unrelated issues. If the commission, after a study of the traffic requirements of the region, had concluded that the loss to existing railways will be greater than the public gain, it had a perfect right to disapprove of the proposed construction; but if it found that the corporate set-up of the railroad is such as to indicate that it is in violation of existing laws, that conclusion should not have affected its decision in the construction matter. A more appropriate course was open to it. It should have called upon the Duke estate and others in control of the railroad to show cause why they should not divest themselves of their transportation interests. In this connection it would be well if the commission were to settle the question once and for all by making an issue of it either in the case of the Piedmont & Northern Railway or in the more conspicuous case of the Ford control of the Detroit, Toledo & Ironton Railroad.

The Human Factor

INCOMPETENT foundation is the term that summarizes the findings of cause of the St. Francis dam failure. It is an apt term, yet it does not fully satisfy. A more fundamental explanation is needed. Was it imperative that the dam be built on this foundation? And why were the numerous safeguards common to good practice neglected? Such questions are inescapable. Answers to them, though not expressly stated, are obvious on full review of the situation. They form the text of one of the great tragedies of engineering practice.

The St. Francis disaster was more than the physical collapse of a great dam; it was the foredoomed failure of a false practice. It was the inevitable outcome of neglecting principles that are well established in sound engineering. Time and again throughout the testimony before the coroner's jury and the other investigating bodies evidence of this appeared. Two facts stood out insistently: First, there had been entire disregard of the need of deep-set foundations, a cutoff wall, pressure grouting or any positive means of excluding water from the foundation as a whole—recognized practices even where the foundation is of the hardest rock, and absolutely vital where the foundation is something less than the best. Second, engineering work in the Bureau of Water Works and Supply always had been dominated by one man, the chief engineer, who took upon himself, in this case at least, entire responsibility, sought no independent opinions and adopted technical policies based on his unconfirmed judgment alone.

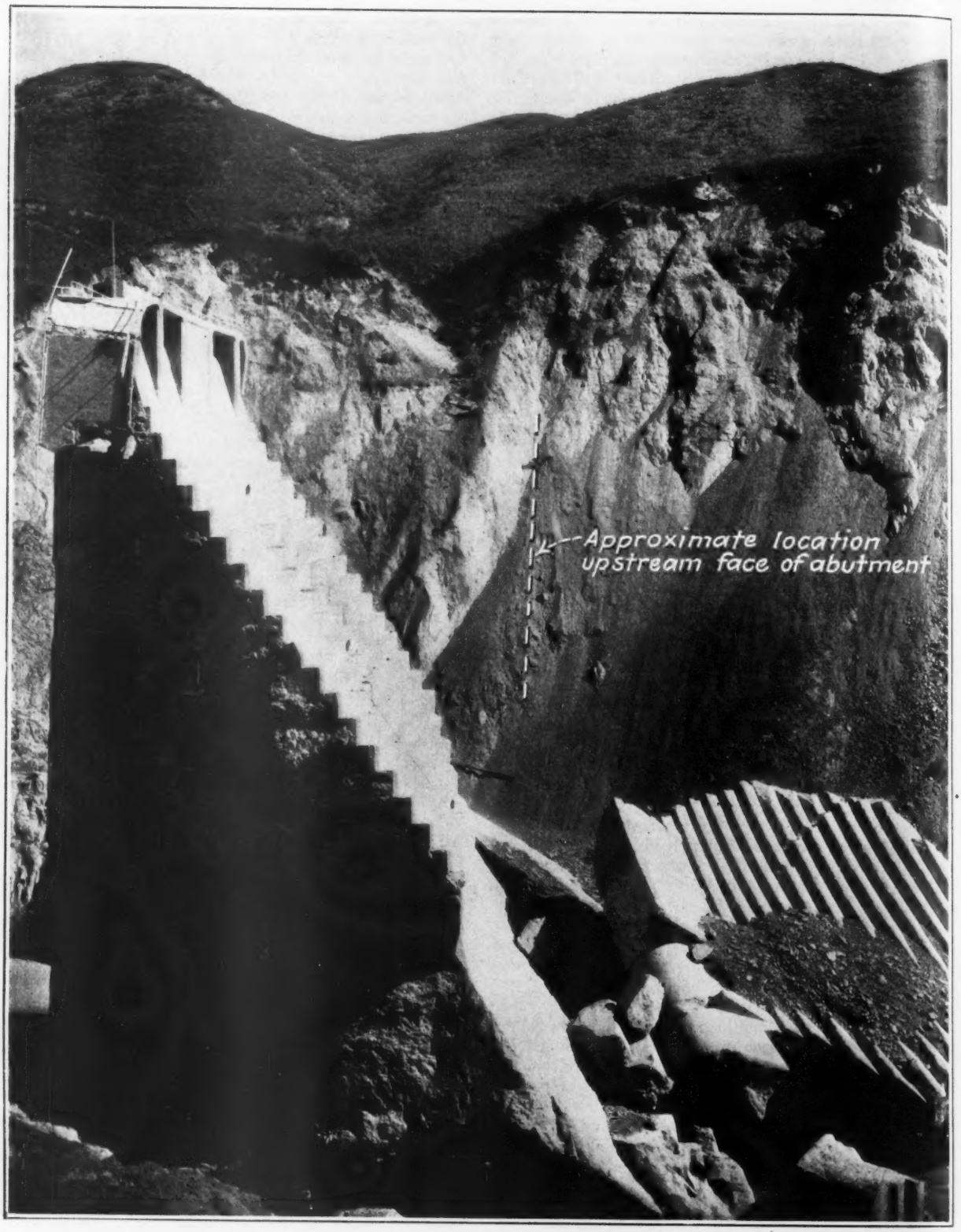
That contraction joints and inspection gallery were omitted may have had nothing directly to do with the actual failure of the structure, but it is further indication of the departure from recognized good practice that has marked the policy in the water bureau. Had the plan of construction used for the St. Francis dam been brought forward by some comparatively inexperienced engineer, or had the work been done by contract or under any other condition that would naturally have brought independent engineering opinion into the case, it is highly probable that some modified plan would have been substituted and the disaster avoided. In other words, if attention had been drawn to the extent to which this structure departed from standard practice in the matter of safety, higher city authorities would have been unwilling to take responsibility for it, even though a change in plans should greatly increase the cost.

But the higher officials had absolute confidence in Mr. Mulholland. The reason that there was no intervention from above and no effective protest from below found specific expression in the statement: "Mr. Mulholland was personally overseeing the work." His own confidence in himself but strengthened the situation—he did not recognize the danger of risking so much on the judgment of one man unsupported by independent opinion or report. Nor can this responsibility pass on to the technical assistants; it properly rests with the head of the department who makes the decisions. He may have information that is unknown to his subordinates, and they cannot go farther than to report their views to their chief. Unfortunately, any such views that were expressed did not have sufficient influence to outweigh the conviction of the head of the department that he was right. Possibly his confidence in the adequacy of the St. Francis foundation was based on his experience in judging foundations for earth and rock-fill dams, in which field he had extensive experience.

William Mulholland, the venerable chief engineer of the Los Angeles Bureau of Water Works and Supply, has set up a record of long and successful service seldom equaled in the engineering profession. For fifty years he has been identified with the water-supply system of Los Angeles. It was his vision, his courage and his energy that made possible the Owens Valley aqueduct, without which the city's phenomenal growth would have been wholly impossible. And never has there been even a suggestion that his work was not honestly and conscientiously done. More than that, he is loved and respected by his associates and by all who know him. Always he has been an indefatigable worker; indeed, he carried on his technical studies at night, poring over his books until late, after days of hard physical labor. For the man who has taken the entire blame for St. Francis dam failure there is only high esteem and sincere admiration!

Yet this very condition brings out in bold relief the truth that the construction of large concrete dams requires more than honest sincerity of purpose and high character. An appreciation of the importance of technical competence and training is as essential in the position of chief engineer as it is in the drafting room where stresses are calculated and designs drawn up. Neglect of recognized safeguards that have become a part of modern practice is to court danger; and to ignore the fatal risk of error that is attached to dependence on one man's autocratic judgment is sure to lead to a day of reckoning.

In view of the gross neglect of essential foundation conditions — geological, chemical and physical — that speaks out of the St. Francis disaster, it is beyond question that future dam construction will give even more thorough study to this element than has been customary heretofore. But with this exception the engineering profession will make no change in methods of design or construction of concrete dams. It is no indictment of the profession that in building one structure some of the engineering principles which the profession has developed and accepted as good practice were entirely neglected. The indictment rather is of the organization policy which made it possible for responsibility for decisions in matters depending entirely upon human judgment to rest in one man. A great price has been paid in human life and property damage for the neglect of this cardinal principle of engineering design and construction. It shall not have been paid in vain if the failure of the St. Francis dam marks the end of the period in which such a policy may go unchallenged.



St. Francis Dam—Standing Portion and Right Bank After Failure

Looking from the left bank across to the right bank past the central portion of the dam, which remained standing. Landslides have obscured much of the rock face. The rock slope should be compared with that shown in Fig. 3.

St. Francis Dam Catastrophe —A Review Six Weeks After

More Data on Design and Construction of the Dam Itself, the Situation in the Path of the Flood Down the Valley and Abstracts of Four Reports of Investigating Committees

By Nathan A. Bowers
Pacific Coast Editor, Engineering News-Record

RESCUE work in the wake of the St. Francis disaster in southern California brought with it more or less hysteria, surging like the flood itself first one way and then another, as public sentiment was swayed by this or that wave of popular fancy. Six investigating committees of engineers and geologists were appointed by as many different interests to investigate the cause of failure, and a remarkably large number of engineers from far and near visited the site. For a few weeks there was too much agitation to gage true conditions accurately. Now, six weeks after the event, a different situation obtains. Many problems remain to be worked out, but there has been a general settling down and clarification that brings certain developments into sharp relief.

Some of these developments are: (1) The incompetence of the foundation materials has been established beyond any reasonable doubt as the cause of the failure. Going a step beyond this immediate cause, there has been put on record an impressive example of the danger in centering great responsibility in one man, no matter what his capabilities, without providing independent opinions to check his judgment. (2) There is in this disaster no indictment of modern practice in dam design, because, quite aside from the matter of the fundamental necessity for testing and proving the quality of the underlying material, this structure lacked deep-set foundations, adequate cutoff wall, foundation drains, foundation grouting or other provisions for preventing water from getting underneath the dam. (3) The celerity with which Los Angeles accepted its responsibility to the devastated area, speedily making available $1,000,000, 2,000 men and the necessary equipment for rehabilitation work, has created an excellent spirit and attitude in the flood-swept valley, where intense antagonism might have developed. Finally, (4) the speed and efficiency with which the A.G.C. organized ten contract units, mobilized equipment and had it working in the stricken area in less than 48 hours after the agreement was signed constitutes a record without parallel in times of peace.

Conditions attending the failure of the St. Francis dam, accompanied by a loss of 426 lives and $10,000,000 property damage, were such that engineers everywhere showed a more intense interest in it than in any other event of engineering interest in recent years. Extra copies of *Engineering News-Record* of March 22, containing Mr. Bowers' preliminary report, were exhausted in a few days. The present article gives a general review of the situation six weeks after the event, and abstracts of reports of four official investigating boards. It thus affords the most comprehensive compilation yet published.

Previous articles on the St. Francis dam in *Engineering News-Record* are:

News paragraphs on construction progress.....March 19, 1925, p. 497
A construction view of the dam.....May 6, 1926, p. 743
News note (with construction view) announcing the failure.....March 15, 1928, p. 456
Report of failure, with pictures and drawings, and editorial discussion.....March 22, 1928, pp. 465, 466
Discussion of geological conditions at dam site.....March 29, 1928, p. 517
Abstract of report of committee appointed by Governor Young.....April 5, 1928, p. 553
News announcement of report findings and letters of discussion.....April 19, 1928, pp. 638, 639

Of the six investigating committees, all have reported except the Ventura County committee. The first report to appear was abstracted in *Engineering News-Record*, April 5, p. 553, and in this issue are abstracts of the four others. In the present article are brought together all important data relative to the dam and its failure that have been brought to light. In connection therewith the rehabilitation work in the valley below is described.

Location and Design—When rapid population increases in Los Angeles brought urgent demand for more water storage at the lower end of the long Owens Valley aqueduct, the St. Francis site was chosen. A dam here would create an excellent reservoir of large capacity in just the desired location. The site and type of dam were selected by William Mulholland, chief engineer and general manager, Bureau of Water Works and Supply. Under the law, no permit or other official approval of site or plans was necessary and the bureau did not of its own volition retain any consulting engineer or geologist. As the construction was done wholly by day labor, the entire operation was under one management without independent check of any sort.

In preparing the designs for the St. Francis dam it was found that the Mulholland dam, a curved gravity type structure built by the bureau in 1924 for storing water above Hollywood, had about the same height and a radius somewhat greater than was required at the St. Francis site. The designs already developed for the Mulholland dam were therefore adapted to the St. Francis site by making the necessary changes. Contrary to the first statement, it now appears that no provision was made for uplift in the design. If the stepping of the downstream face was continued to the bottom of the excavation, which seems to have been the case, the total base width cannot be considered effective in calculating strength against overturning, because a distance back from the toe sufficient to develop a shearing strength equal to the toe load would have to be allowed.

The St. Francis plans were made to include a low

extension or dike some 600 ft. long on the right bank, also a spillway with a capacity of about 2,700 sec.-ft. consisting of openings 18 in. deep beneath the roadway in eleven of the 23 bays.

On the right (west) bank several test pits were sunk, and one was filled with water as a test of permeability. The left bank was too steep for test pits, but an exploration tunnel was driven in about 30 ft. without finding any change in the laminated schist appearing on the surface. Core-drill holes put down in the streambed (referred to later) were not made for purposes of exploration but were intended rather to provide drainage and to avoid danger of uplift.

Construction—Construction was begun in August, 1924. Overburden was sluiced off with a hydraulic giant, and then a power shovel did most of the excavation in the streambed and in benches on the right bank. Where material could be loosened beyond the depth to which the power shovel went, this material was taken out with picks and gads. As the left bank was too steep for the shovel, only hand work was done there. No explosives were used anywhere in the foundation. Excavation in the channel was carried 30 ft. below streambed and in the side walls 10 or 12 ft. into the rock. Final excavation of the foundation trench in the canyon walls was not done all in advance, but was completed a few feet at a time just ahead of the concrete pouring.

When excavation began in the streambed, the first step was to build a combination cofferdam and temporary diversion, consisting of a massive concrete wall with its upstream face in the same plane as the face of the dam, thus making it possible to incorporate this concrete within the structure. This wall was carried down about 20 ft. below streambed.

With streamflow thus diverted, excavation was carried on behind the wall and was continued down to a level 8 ft. below its base, or 30 ft. below streambed. In going to a level below the wall, however, no material was excavated beneath the wall, thus leaving a setback 8 ft. deep in the heel of the maximum section. Before pouring concrete in the streambed, eleven 4-in. core-drill holes were put down 15 to 30 ft. farther. These were solely for drainage purposes, and their tops were fitted with pipes that joined within the concrete and delivered to a common outlet extending through the dam.

Seepage—The dam was completed in May, 1926, two months after storage had begun. In 1927 the reservoir was nearly filled, but it did not remain at that level (see storage record, Fig. 8). The failure on March 13, 1928, occurred almost immediately after the first complete filling.

FIG. 1—UPSTREAM VIEW OF WRECKED ST. FRANCIS DAM

Note general character of reservoir and surrounding topography. Contact between schist (below) and conglomerate (above) in west bank of canyon is marked *A*. Above this contact, at point *B*, there is a small pool, possibly the location of the initial failure. The canyon side generally was scoured away to a depth of 20 ft. or more.

As soon as storage began, moderate seepage appeared on both sides of the main dam and on the slope of the spur below the long westerly dike. This latter seepage became so troublesome that finally a line of drain tile was buried along the downstream face of the dike. Immediately prior to the break work was under way on this hillside in building a new road made necessary by the frequent slides due to water seeping out on the bank of the old road near the dam.

At all times more than half the total seepage came from a point just above the contact between schist and conglomerate, according to testimony, and the material increase of seepage (estimated at 1 sec.-ft.) that was observed twelve hours prior to the break was practically all from this one point. In other words, the total seepage from all points on the afternoon before the break is estimated to have been 2 sec.ft., or twice the total observed at any previous time.

The total seepage is known to have been less than 10 sec.-ft. as late as 30 minutes before the break, when several automobiles passed over the canyon

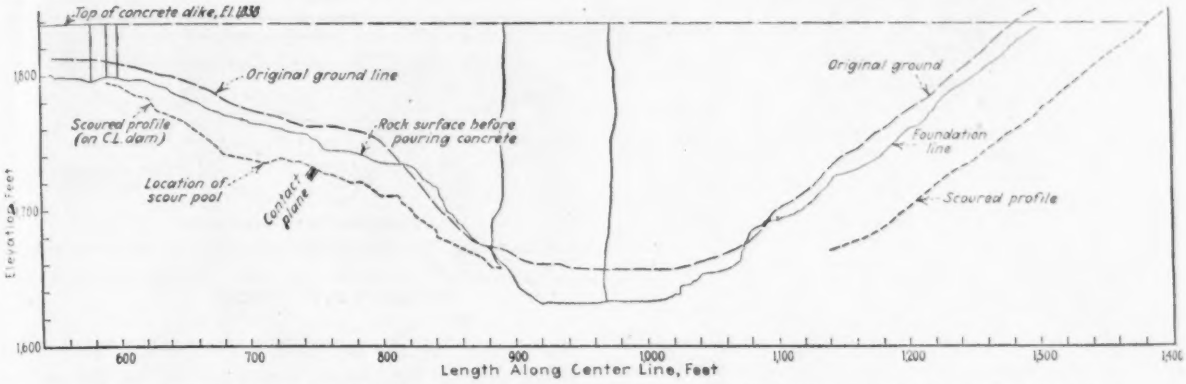


FIG. 2—ORIGINAL AND PRESENT CROSS-SECTIONS OF SAN FRANCISQUITO CANYON AT SITE OF DAM
Roughness of the original stadia survey from which the ground line was plotted is assumed to be responsible for the obvious errors which place the ground line below the dam foundation line at two points.

road leading to the dam. This road many times crossed the streambed through which seepage had to pass, and 1½ miles below the dam it crossed a culvert of 10 sec.-ft. capacity in a shallow fill. Had the culvert capacity been exceeded, the road would have been flooded.

All five reports are in entire accord in finding that incompetent foundation material was responsible for the failure; several suggest that the inception of the break was probably in the softened conglomerate above the contact plane. Because a scour pool remained at this point, even after the 15 or 20 ft. of erosion due to escaping water, there is some support for the theory that the foundation blew out here first, the high velocity of escaping water cutting a deep hole not entirely obliterated by the subsequent general erosion.

Shortly after the break a sample of water seeping from a ravine on the right bank just below the dam was taken by S. B. Morris, chief engineer, Pasadena Water Department, and a chemical analysis of this sample was compared with an analysis of a sample of aqueduct water taken from the surge chamber of Power House No. 2. The seepage was found heavily charged with calcium and magnesium sulphates, silicates and chlorides, as would be expected in water that had dissolved gypsum. The results of the analyses were as follows:

	Aqueduct Water at Power House No. 2 Surge Tank P.P.M.	Seepage, St. Francis Dam Site, Right Bank, P.P.M.	Ratio of Second Column to First Column
Chlorides	24.00	241.50	10.1
Evaporated solids	294.60	2,319.00	7.9
Silica	35.20	666.20	18.9

Foundation Materials—The geological study made by Messrs. Hill, Murphy and Tolman included a classification of the material in the conglomerate, made by the static elutriation method, with results shown in the accompanying table. Their investigations also found that seepage through the conglomerate had averaged not less than 2 miner's inches (0.04 sec.-ft.) since storage began. At this rate and assuming a solution of 0.002 per cent, 97 tons, or 1,348 cu.ft., of gypsum could have been dissolved and carried away from the gypsum veins found in profusion in the conglomerate. Some of the gypsum veins are 2 in. thick, and as much as 2 per cent of the rock in the fractured zones was found to be gypsum.

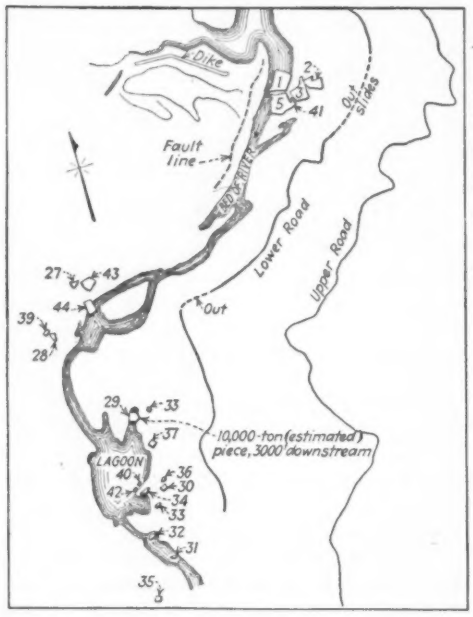
Water containing gypsum in solution is clear. Obviously the clearness of seepage through rock may not be taken as evidence that no elements essential to the tightness of the formation are being leached away.

In the light of these findings it is conceivable that gypsum was dissolved by seepage first where water entered the foundation at the upstream face. The rock structure might then have been progressively weakened as gypsum was leached from the veins farther and farther under the dam, with attending softening due to water absorption. Other material broken out by the release of soluble minerals might readily lodge again farther in where only a solution could filter through. A weakening that progressed in this way might not be accompanied by any great increase of seepage, and only when the dissolving process by slow degrees had gone entirely through the foundation would the final breaking down of the rock structures occur.

MECHANICAL ANALYSIS OF SANDSTONE FROM THE CONGLOMERATE

Material	Percentage	
Sand over 1 mm	1.26	
Sand 1 to 0.5 mm	6.69	
Sand 0.5 to 0.25 mm	9.17	
Sand 0.25 to 0.12 mm	31.43	
Sand 0.12 to 0.05 mm	24.11	
Total sand		72.66
Coarse silt 0.03 to 0.05 mm	7.19	
Medium silt 0.05 to 0.015 mm	6.16	
Fine silt 0.015 to 0.005 mm	6.87	
Total silt		20.22
Particle clay (retained on filter paper)	4.04	
Colloidal clay (passes filter paper)	1.88	
Total clay		5.92
Moisture expelled at 110 deg. C	1.19	1.19
Unaccounted for	0.01	0.01
	100.00	100.00

FIG. 3—LEFT ABUTMENT DURING CONSTRUCTION OF DAM
Shows the surface of the schist laminations against which concrete was poured. Forms at right are for stepped downstream face of dam.



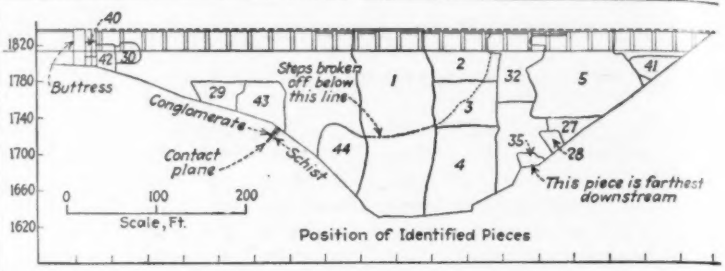


FIG. 4—LOCATION OF FRAGMENTS, AND ORIGINAL POSITIONS OF IDENTIFIED PIECES

The Break-Up of the Dam—Wherever the initial break is assumed to have occurred, the first block of concrete to move out was probably in contact with the foundation. Any cantilever action or arching of unsupported parts of the dam would concentrate excessive loads and moments on such parts as remained in place. Such excess stresses must have occurred at least momentarily in the section that still remains standing. On the other hand, when these stresses came it is probable that the effective height of water pressure behind the dam had been materially lessened, because, even if there had not yet been time to draw down the reservoir, there would have been a material drop in level close to the break caused by velocity head. Evidence of the steep downward curve of the escaping stream appears in the flood mark on the right bank just below the dam, where a man might have stood safely all through the break-up, at a point 70 ft. below the full reservoir level.

Scour and Movement—Resurvey of the site showed that erosion on the left bank has extended 55 to 60 ft. below original foundation levels, and at that the surface is so covered by sliding material that the actual rock face cannot be determined. Renewed speculation regarding the sequence of the failure was aroused when piece 35 from the left bank was identified as the one farthest downstream. This is the more puzzling because all large fragments on this side remained within the breach.

In the process of identifying the fragments of the broken dam, fragments on which the downstream steps remain were easily located, at least as to original height in the dam, because each terrace has a different width.

Piece 43 had adhering to its upturned base schist, black gouge and conglomerate, all in close proximity, thus locating it definitely as coming from the contact plane in the right bank.

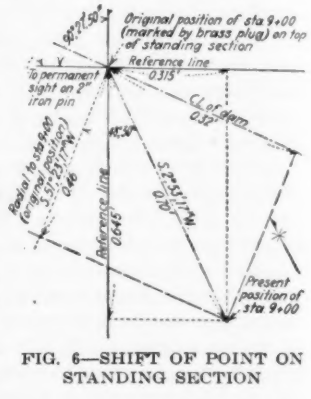

Original position of sta 9+00 (marked by brass plug) on top of standing section
Reference line
To permanent sight on 2" iron pin
Present position of sta 9+00

FIG. 6—SHIFT OF POINT ON STANDING SECTION

A resurvey after the break tying in to a hub on top of the standing portion indicated (Fig. 6) a displacement of 0.7 ft. Two other points located part way down on the downstream face indicated lesser movement here, which tends to show that such slight change in position as there was consisted of twisting and tilting rather than any sliding on the base. This is still a moot question; one geologist claims to have found indication of sliding at the base of the standing portion. In view of the terrific wrenching evidenced by the two sheared faces, however, it is remarkable that more movement did not occur.

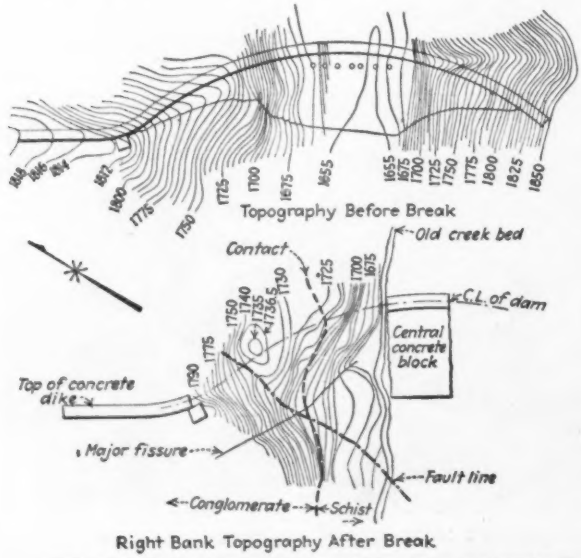


FIG. 5—WEST BANK TOPOGRAPHY AT DAM SITE, BEFORE AND AFTER

While both ends of the standing block show fresh breaks, there are also large darkened areas of irregular shape, believed to be the location of shrinkage cracks that had occurred previously. The 100-ft. length of this block is not an unusual distance between the shrinkage cracks that occur in such structures if contraction joints are omitted, as was the case on St. Francis dam.

Aid and Rehabilitation Work—For a few days after the disaster the first thought was saving life and recovering bodies. Then rehabilitation problems were given attention. To the relief committees assigned to the sticken area, H. A. Van Norman, assistant engineer of the Los Angeles Bureau of Water Works and Supply, stated clearly a very definite policy: "The city built this dam to provide storage which it was obliged to have. No question of the legal status of claims should ever be raised; the moral obligation to repay damage in the

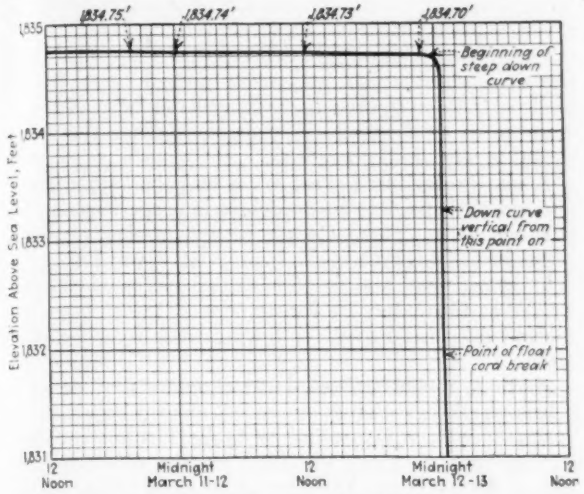


FIG. 7—COPY OF GAGE RECORD

Gage time probably does not correspond exactly with correct time.

valley is sufficient." The city officials made every effort promptly to aid those who had suffered damage. For emergency work in the valley the city council made an initial appropriation of $1,000,000 to be spent on authority of the city attorney working through a citizens' advisory committee, with Mr. Van Norman as chief engineer. Ventura County authorities likewise selected an advisory committee of seven to co-operate with city officials, and a speedy but systematic survey of the stricken area was made.

For a 40-mile length of the Santa Clara Valley from San Francisquito Canyon to the sea, lands were denuded by the flood and a strip from 1 to 3 miles wide had been made a desert. Where there had been orchards and fertile farm lands are now wind-swept stretches of bare silt and gravel deposits. Enormous quantities of débris had collected around trees, against buildings and in great

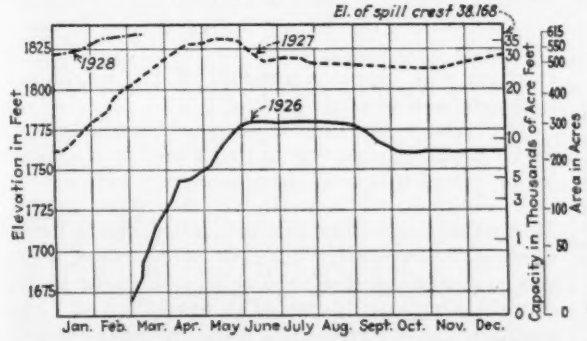


FIG. 8—RECORD OF STORAGE IN ST. FRANCIS RESERVOIR

piles along the margin of the denuded area. Some 40- or 50-acre orchards were literally buried under continuous piles of débris from 2 to 6 ft. deep.

The devastated region included citrus orchards valued at $1,200 to $1,500 per acre. Where orchard trees were not actually destroyed, there was time to save them by prompt removal of débris. In the general survey aided by farm advisory officials of the state a priority rating of 1 was given to such areas. Other damage was given priority ratings of 2, 3 or 4, according to the degree to which restoration was of an emergent character. Printed "authorization" forms were prepared, and before work was begun on any tract the land owners signed an agreement that the cost of work so done should apply on property damage claims against the city. By speedy moves in getting this work under way it was possible to prevent the far greater damage sure to result from delay. Each authorization was numbered, and each work order was known by this number throughout the records of city, contractors and county.

The A.G.C. Makes a Record—The Southern California Chapter of the Associated General Contractors was early recognized by the city authorities as a ready source of equipment and organization needed for débris removal. On request of the city, A.G.C. officials selected ten contractors able to dispatch crews and equipment to Ventura County at once, and an agreement was drawn up under which camps and commissary were furnished by the city, the contractors furnishing men and machinery. This agreement was signed on a Saturday, and by noon on the Monday following $1,500,000 worth of equipment and 1,300 men had been moved to the scene, 60 to 80 miles from Los Angeles, and every one of the ten contracting units was at work on an assigned job.

The city let the commissary contract to the A. B.

FIG. 9—ORANGE ORCHARD DESTROYED BY FLOOD FROM DAM

Each tree caught its own pile of débris. Great masses of mud were brought down and deposited by the water.

Anderson Company, Los Angeles, at 10 a.m., and at 6 p.m. of the same day a commissary unit to feed 400 men had been installed at field headquarters and dinner was being served in a large tent.

Main camps for the ten débris-removal units were established at Santa Paula, Fillmore, Bardsdale and Piru. Operation was under the organization scheme shown in the accompanying diagram, Fig. 10. The maximum number of men at work was 2,000. Each contractor, with 100 to 300 men and perhaps fifteen pieces of equipment, was assigned a letter, and each piece of equipment was

EQUIPMENT CLEARING UP DÉBRIS ON APRIL 19

Type of Equipment	No. in Use
Ford dump trucks	66
Trucks	44
Caterpillars	95
Cars	36
Power saws	4
Clamshells (on crawlers)	12
Rotary fresnos	27
Loaders	2
Smeisers (a wide scraper)	3
Graders	8
Scarifiers	7
Disks	9
Oil spray trucks	2
Slip scrapers	18
Backfillers	4
Truck and low bed trailer	1
Steel sleds	60
Total pieces	398

numbered. A record of the time for each machine was kept by card index, and work under each authorization number was checked twice daily by timekeepers of both city and contractor. Pay checks for the seven-day week ended on Saturday were ready for the contractors not later than the following Monday.

While each of the ten contractors organized his work as an independent unit, the control was such that equipment was frequently requisitioned and transferred from one unit to another as conditions required. Speeding up such interchanges was the duty of a transportation superintendent, for whose use was provided a heavy-duty trailer.

A $350,000 Débris-Removal Job—The major portion of the work assigned to the ten units consisted of removing, piling and burning débris, leveling off soil deposits, scraping roadways, excavating buried autos and other property, removing deposits of injurious depth around orchard trees, etc. However, work orders called for "restoration," and contractors interpreted this literally by tackling every restoration problem presented. For example, due to the breaking of three 8-in. oil pipe lines from the oil fields a large quantity of oil had floated down and left a coating on trees in valuable orange orchards. It took some experimenting to find a means of removal not harmful to the trees. Spraying machines were tried, but cleaning by hand with scrubbing brushes was ultimately found best.

The débris piles did not burn readily, as they contained much wet earth and most of the wood was green. After proper piling, however, most of the piles could be fired with portable kerosene preheating torches. For the worst cases oil spray trucks were used which applied oil under high pressure delivered through 100-ft. lengths of hose. Thus deep oil penetration into the piles was secured and a very intense heat could be developed.

For moving piles of light débris, crawler-mounted grab-bucket machines with long teeth proved best. Hooks made of rails were used for moving large logs. Tractors were in great demand everywhere, because they were useful in almost all operations.

A simple device of great value was the "stone boat," or steel sled, used to move débris in large loads to the burning piles. These sleds were made of $\frac{3}{16}$-in. sheet metal in 6x12- or 6x14-ft. sizes, bent upward for 6 or 8 in. at each end and fitted with two pieces of old rails welded on the under side for runners. Chains fastened to the rail ends were used for towing these sleds behind a tractor.

At first, rehabilitation of the thousands of acres of débris-wrecked orchards appeared almost hopeless. It would have been impossible with the farmers' methods and equipment. Early estimates put the débris-removal cost at $600,000 and the time at 90 days. The A.G.C. committee cut these estimates to $335,000 and 30 days, and when the writer visited the work on April 18 the prospect was that completion would be well within the latter figures. The farmers were surprised and delighted with results, and everywhere satisfaction and a good spirit prevailed over the success in débris removal. Great credit is due the A.G.C. members for the spirit and skill that were prime factors in this work, particularly because no profits were figured, the contract being on a cost basis with 15 per cent added to labor for overhead.

Rehabilitation of wells, irrigation systems and pumping plants was done by the city's own forces, an allotment of $150,000 being made for this work. City crews also did such survey and engineering work as was required. For example, in Santa Paula it was desirable to establish the probable limits of future high water along the river, whose channel had been considerably raised by flood-deposited material. Fourteen cross-sections of the present stream-bed were therefore surveyed, and markers were set at flow line for floods of 120,000 and 145,000 sec.-ft., the former being rated as probable once in 25 years and the latter once in 100 years. Calculations were based on a runoff of 100 sec.-ft. per square mile of drainage area.

The reconstruction of buildings, which will be handled in much the same way as the débris removal, could not be started so speedily because owners were slow to make decisions. This work may take three months. Most of the building reconstruction is in Santa Paula, where 427 houses were damaged or destroyed. The estimated total number of buildings damaged in the entire flooded area is 600. On April 23 Red Cross reports placed the number of bodies recovered at 289, persons still missing at 137 and dependents at 27. A rough estimate of total damage exclusive of death or injury claims is given in the accompanying table.

The source of funds to meet the damage claims has not yet been determined. At first city officials favored an increase in water rates such as would net about $2,500,000 per annum. The rate increase was vigorously opposed, however, and as this is written the popular

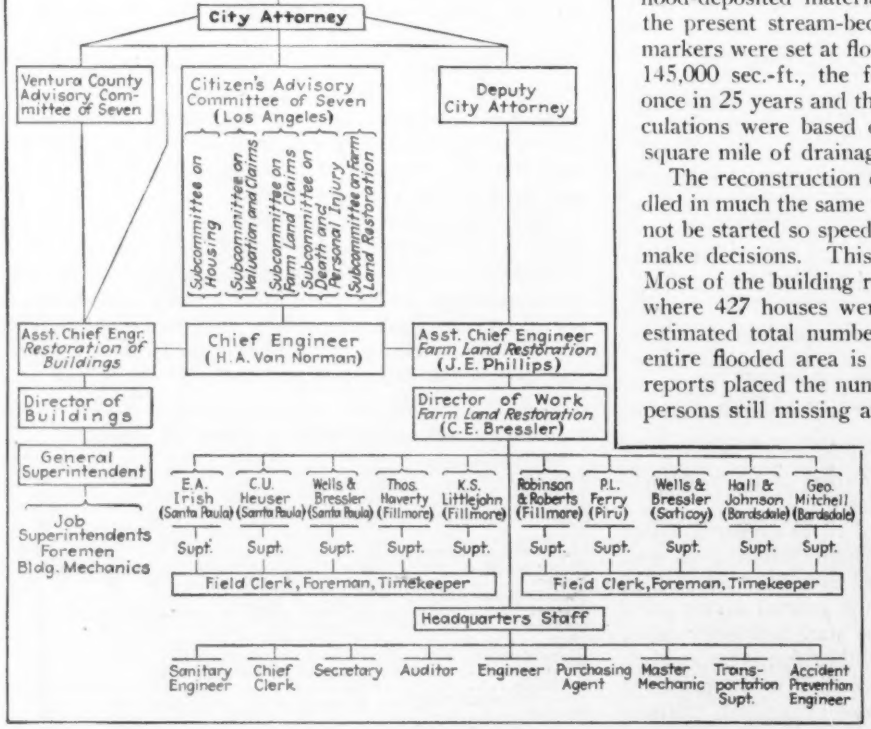


FIG. 10—ORGANIZATION OF REHABILITATION WORK

demand is that city funds exceeding half of the estimated damage which have been allocated to the development of municipal power should be applied as a first payment. The point is that the city apparently has accepted full responsibility and is concerning itself with some feasible plan of raising money to pay the bill.

Mulholland Dam—Since the St. Francis failure attention has been focused on the Mulholland dam above a very thickly settled section of Hollywood. Work on this structure was started in August, 1923, and completed in December, 1924. It has a gravity section, a height of 204 ft. and a crest length of 925 ft., is arched in plan on a 550-ft. radius and has a storage capacity of 7,500 acre-ft. Allowable foundation pressures were taken as 12 tons per square foot at the heel and 10 tons per square foot at the toe. The formation is reported as a good quality of sandstone both in bottom and sides. Foundation excavation was carried into the rock about 12 ft. No allowance was made for uplift, drain pipes being expected to relieve any hydraulic pressure beneath the structure. No contraction joints were put in the dam, and as contraction cracks have occurred they have been grouted.

The structure has no spillway, because there is neither drainage basin above nor stream channel below—the reservoir being really a tank in which to store a water supply for local use. This reservoir has the remarkably short high-water length of 3,600 ft. The supply flows in by gravity from the San Fernando reservoirs, supplemented by pumping from San Fernando Valley wells. The reservoir was first filled in 1925 and has been full several times for considerable periods. The maximum seepage through or around the dam is reported by city officials never to have exceeded about 10 miner's inches (0.2 sec.-ft.). Various dams of the Water Bureau and particularly the Mulholland dam will be the subject of investigation by two committees, announced in *Engineering News-Record*, April 26, p. 677.

ESTIMATED DAMAGE RESULTING FROM THE ST. FRANCIS DISASTER
(Exclusive of Death and Personal Injury Claims)

Damage to land, crops, wells, etc.		$5,000,000
Southern California Edison Co.		500,000
Buildings (chiefly in Santa Paula)		600,000
Highways and bridges		400,000
Personal property		250,000
Southern Pacific railroad		175,000
Southern California Gas Company		50,000
Total for the valley	$6,975,000	
St. Francis dam	1,325,000	
38,000 acre-ft. water at $7 per acre-foot	266,000	
Restoration work on Power Plant No. 2	750,000	
Total		$9,316,000

Reports of Four Boards of Engineers and Geologists

All Are in Complete Agreement on Finding That Foundation Failure Was Responsible for Disaster

AGREEING fully in their main findings, four technical boards or juries which investigated the St. Francis dam failure of March 12 attribute the disaster to defective foundations. Three of the boards were made up mainly or wholly of engineers, and these specifically place the origin of the collapse on the west side of the canyon, where unconsolidated foundation strata were softened by the water and either squeezed out by the pressure or washed out by an enlarging leak. The fourth board, made up of geologists, does not go into the engineering side of the matter to this extent.

All the reports intimate, and one (the district attorney's board) explicitly states that no dam should have been built at this site. Another, the coroner's jury, names a number of defects in construction. Either in terms or by inference, several of the reports place the ultimate responsibility for the accident on dependence on the unconfirmed judgment of one man, the chief engineer, William Mulholland. Reform of state legislation is demanded, in order to place municipal dams under state check of design and construction as required for private dams. Essential parts of the four reports follow.

Geologists to Los Angeles Water Board

A BOARD of three geologists, Dr. Robert T. Hill, Prof. C. F. Colman and D. W. Murphy, appointed by the Los Angeles Board of Water and Power Commissioners, found that the foundation rock was defective. Its full report is not available, but a summary of findings was issued by the board, parts of which follow.

The manner in which the dam failed or where the first break occurred has not been determined.

There are no records of any seismic disturbance that could have caused the failure.

There were weaknesses in the foundation under both the east-

FIG. 11—A RECONSTRUCTION CENTER
The commissary contract was let at 10 a.m., 60 miles away. The tent was up and dinner was served at 6 p.m. of the same day; seating capacity 300.

FIG. 12—PILING DEBRIS READY FOR BURNING
Thin deposits of débris were gathered and piled by hand for burning. Heavy drifts were gone over and moved first in the search for bodies, and later again moved to be piled for burning.

ern (left) and the western (right) abutments sufficient to have caused failure. The weakness of the eastern abutment was due to existence in direction of the slope of the planes of schistosity. These planes, especially when wet, would permit easy sliding under the abutment. The weaknesses on the west side were due to the inherent character of the foundation material. The cementing material in the conglomerate consists of clay which softens when wet and thus reduces the strength of the foundation material. The conglomerate was cut by numerous fissures filled with gypsum. This gypsum is slowly soluble in water and any passage of water under the dam, even in small amounts, would in time produce openings. The dissolving and carrying out of the gypsum would produce a condition whereby the rock masses softened by water could be carried out of the foundation.

Recent surveys show a small movement at two points on the right bank (west side) since the dam was constructed. It is possible that local movement of the conglomerate above the fault line may have been a contributing factor in starting the break.

District Attorney's Technical Commission

REPORTING to the district attorney of Los Angeles County, a commission of engineers and geologists (E. L. Mayberry, Walter G. Clark and Charles T. Leeds, engineers; Allan E. Sedgwick and Louis Z. Johnson, geologists) ascribes the failure to giving way of the foundation under the west wing when the rock formation there became saturated with water from the reservoir. An exhaustive review of the geology of the site and data on laboratory tests of rock and concrete samples are given in the report, dated April 4. The site is declared unsuitable for dam construction, but the "examination and understanding" of the foundation materials before construction were not sufficiently thorough.

The design of the dam was in accord with accepted practices. Examination and tests of the concrete indicate that the quality was satisfactory.

The slides now in evidence above and below the dam site were caused by the movement of water cutting out the talus at or below the water level, and allowing the earth and rock to slip to a natural angle of repose. The board has been unable to find evidence to support a theory that the destruction of the dam was or might have been caused by an explosion, either accidental or intentional.

Failure of the dam was due to defective foundation materials, some of which, while reasonably hard when dry, became soft and yielding when saturated with water.

The defective foundation material referred to herein as conglomerate became softened by absorption and percolation of water from the reservoir and was by hydrostatic pressure pushed out from under the dam structure, permitting a current of water at high velocity to pass under this sector of the dam. This current, by eroding the soft foundation material, quickly extended the opening under this portion of the structure to such an extent that a part of the westerly sector of the dam collapsed through lack of support. Following failure in the westerly sector, the escaping water swirled across the downstream toe of the dam to the easterly side of the canyon, cutting away the easterly wall of the canyon below the easterly abutment, causing a slide which broke the already weakened bond between the easterly sector and the side wall of the canyon, allowing the easterly sector of the dam to collapse.

The record made by the automatic recording gage on the dam shows that from midnight on March 11 to 11:30 p.m. on March 12 the water level in the reservoir dropped 0.05 ft. The Bureau of Water Works and Supply reports that water was not drawn from the reservoir during this period. This change in water level amounts to 1,335,114 cu.ft. which passed out of the reservoir during 23½ hours, or at a rate of 12.9 sec.-ft. (corrected for evaporation). The drop of water level in the reservoir from 8 p.m. March 12 to 11:30 p.m. March 12 was 0.035 ft., or 934,580 cu.ft. during 3½ hours, or at the rate of 74 sec.-ft. The movement of the gage after 11:30 is so rapid that it cannot be accepted as an indication of the water level in the lake, but merely as an indication of the water level at the face of the dam.

It is the conclusion of this board that the dam was constructed without a sufficiently thorough examination and understanding of the foundation materials upon which the dam was constructed; that failure of the dam was due to the incompetent geological formations upon which the dam was constructed; that the dam as designed should not have been constructed at this location, and that due to local conditions it is not economically feasible to erect a safe dam at this location.

With reference to the rock formation along the fault line, the report states:

The first foot or two of the conglomerate at the contact shows a decided flow structure where movement of the conglomerate has taken place over the underlying shale. This movement has produced a shear zone along which water has moved with relative ease, disintegrating the conglomerate at the base. . . . Near the surface, underneath the buttress of the wing dam, the conglomerate is so disintegrated that it has nearly lost its original character and is so soft that it can be excavated easily with a prospector's pick and in some places by hand. . . . When the reservoir was filled, the many channels through the conglomerate permitted the water to saturate it. The progressive movement of the water from the reservoir through the conglomerate carried the clay and some of the sand and iron oxide with it, leaving a spongy, cellular structure incapable of carrying great loads.

On the evening of the failure, as indicated by the gage record, a large leak developed, probably not far from the westerly abutment that remains standing, and undermined the dam for a length of 120 ft. or more. "The tremendous weight of the dam over this section caused it to shear off, probably at a shrinkage crack similar to those shown on either side of the center standing section. This left the pressure of the water in the reservoir resisted by two cantilevers, one extending easterly from the westerly standing abutment and the other extending westerly from the center standing section. The cantilevers failed progressively in sections."

Slides along the easterly wall of the canyon are ascribed to the movement of water after the collapse of the dam.

Recommendations—The report recommends that a state permit be required for the construction of any dam, and that a board of engineers and geologists appointed by the Governor pass upon the design, foundation, conditions and construction of all dams constructed in future.

Verdict of the Coroner's Jury

THE coroner's jury was selected with a view to the engineering nature of the evidence that it would receive. Its members were Irving C. Harris, consulting engineer, foreman; S. C. Lines, industrial engineer; Blaine Noice, consulting engineer; O. G. Bowen, engineer and contractor; C. D. Walz, engineer; William H. Eaton, engineer and contractor; H. G. Holabird, appraiser; R. F. Ware, engineer and contractor, and Z. N. Nelson, engineer.

After exhaustive investigation the jury on April 12 reached a unanimous verdict, which gave a comprehensive statement of facts, charged the disaster to foundation failure, pointed out contributing defects in the dam, declared the responsible cause to be reliance on one man's judgment, and called for legislation to prevent the recurrence of such a condition.

Conclusions—The St. Francis dam was defective due to the very poor quality of the underlying rock structure upon which it was built and to the fact that the design of the dam was not suited to inferior foundation conditions. The actual failure was caused either wholly or in part by these defects.

The construction of this dam, without having the design and foundation conditions passed upon by independent engineers and geologists, and without more thorough and systematic methods of design, supervision and inspection, involved an error in engineering judgment in determining the character of the foundations, and an error in fundamental policy relating to public safety. The responsibility for the error in engineering judgment rests upon the Bureau of Water Works and Supply and the chief engineer thereof. The responsibility for the error in public policy belongs to those to whom the chief engineer is subservient, including the Board of Water and Power Commissioners and the legislative bodies of city and state, and to the public at large.

A sound policy of public safety demands that the construction and operation of a great dam should never be left to the sole judgment of one man, no matter how eminent.

The exemption of municipalities from supervision by state authorities in the building of dams involving public hazards is a very serious defect of the state law that should be corrected.

Defects in the Structure—The destruction of this dam was caused wholly or in part by the failure of the rock formations upon which it was built and not by any error in the design of section of the dam itself or defect in the materials of which the dam was constructed. The gravity section accorded with standard practice and would have produced a safe structure if it had been built upon hard, impervious rock, as was supposed to be the case by those who built it.

The dam was in a defective condition due to the following:

1. The dam was built upon two formations meeting at a fault contact, which naturally caused a line of weakness across the axis of the dam.

2. The schist upon which about two-thirds of the mass of the main dam rested is a weak material, badly shattered, very susceptible to seepage of water and to slippage along the planes of cleavage.

3. The dip and strike of the planes of cleavage of the schist were at very unfavorable angles with reference to the lines of pressure of the dam and very favorable for landslides on the east wall of the site of the dam.

4. The conglomerate upon which the western portion of the dam was built is variable in structure, badly seamed in several directions and very deficient in bonding material. While it has the appearance of rock when dry, it is weak in compressive strength; and when saturated with water it disintegrates into a slippery mass of clay, sand, small pebbles and other included materials.

5. Gouge materials at the contact between the two formations were inferior in strength to either or the two principal formations, causing a zone of weakness that can always be expected at such contacts.

6. In addition to the contact fault there is another fault crossing the dam transversely and intersecting the contact fault about 150 ft. downstream from the toe of the dam.

7. The dam was not carried far enough into the bedrock and had no cutoff walls.

8. There was no reinforcement or blanketing with concrete at the contact between the dam and the rock abutments, as is usual where the formations are not of the best. No pressure grouting of the bedrock was done.

9. The dam was not provided with inspection tunnels with drainage pipes, discharging separately into the inspection tunnels for the purpose of locating any leakage and grouting it off if necessary. The only drainage pipes installed were confined to the center portion and connected to one manifold and outlet, making it impossible to localize leakage.

10. The dam was built without predetermined expansion joints. This is a debatable question; but the best practice at present calls for their use.

Methods of Design and Construction—The exploration work and testing of foundation materials prior to reaching a decision to build a dam at this site were entirely inadequate to definitely determine the true qualities of the bedrock and the proper type of dam to be built at the site. . . . . Inspection of the work during construction was very limited and defective in principle owing to the fact that the inspections were made by the same people who were directing or doing the work, without adequate check inspection by an independent personnel to reduce the effect of personal error. Extensive exploration work, inspection and testing of materials and consultations with experts unquestionably would have revealed the true character of the site and prevented this calamity.

Leakage—The total leakage that was observed at any time up to within a few hours of the failure was remarkably small, considering the critical condition existing under the dam, and less than is commonly present in dams that are considered safe and have stood for years.

There have been reports by subordinates and others who claimed to have observed increased leakage and muddy water coming from the dam a few days before the failure. In consequence of these reports, the chief engineer and his principal assistant inspected the dam about twelve hours before the failure and found that the water was coming out clear but picking up surface soil in flowing from the points of first appearance to the afterbay pool below the dam. This agrees with the testimony of all witnesses who made observations at sufficiently close range to see the water where it first emerged from the ground.

The amount of leakage and its character were not such as to cause apprehension of those in charge of the dam and to cause them to doubt their previous faith in the safety of the structure. If they had concluded that the dam was in danger of destruction, the only safety measure possible at that time would have been to warn everyone in the possible path of the flood to move out, since, with all gates wide open, it would have taken many days to have drawn the water low enough to remove the danger.

There is ample evidence to show that no alarming symptoms were observed up to within 30 minutes of the first break, and that the entire destruction of the dam was completed in a very few minutes.

Personnel—The entire personnel of the Department of Water Works and Supply and the Department of Water and Power Commissioners, who were responsible for the building of the dam, appears to have had an unusual degree of confidence in the chief engineer and relied entirely upon his ability, experience and infallibility in matters of engineering judgment.

With a background of many years of very distinguished achievement in the building of a great water-works system, including seventeen earth-fill and rock-fill dams, there was ample reason for a very high degree of confidence.

However, the chief engineer and his principal assistants had had limited experience in the building of large masonry or concrete dams prior to the building of the St. Francis dam. Earth-fill and rock-fill dams are built wholly or in part on unconsolidated or yielding foundations, while masonry or concrete dams must have hard, impervious, unyielding foundations. This organization apparently did not appreciate the necessity of doing the many things that must be done in order to be certain that the foundation of a dam of the concrete gravity type is and will remain hard, impervious and unyielding.

As a consequence of these conditions, serious errors were made, while the entire responsible organization seemed confident that they were maintaining their previous high standard of accomplishment.

There appears to have been no disposition to neglect anything because of expense, as funds were available to meet any requirement deemed necessary. The only reasonable explanation is that those in charge were completely deceived as to the true conditions and acted through ignorance and not from intent to omit any precaution from any motive.

Engineering Committee of the City Council

THE city council of Los Angeles asked Dr. Elwood Mead, chief of the U. S. Bureau of Reclamation, to serve as chairman of a committee to report on the St. Francis disaster, the two other members of the committee to be selected from names suggested by the American Society of Civil Engineers. Lansing H. Beach and Louis C. Hill were chosen to serve with Dr. Mead and in addition to signatures of these three, the report was also signed by D. C. Henny, consulting adviser to the committee, and R. F. Walter, chief engineer, U. S. Bureau of Reclamation. This report, submitted on March 31, agrees fully with the other reports in finding the type and dimensions of the dam and the quality of the concrete adequate, denying that seismic movements caused the failure, and placing responsibility on defective foundations.

Rock Conditions—The mica schist on the left bank and in the streambed is dealt with at some length. The schist is "so thoroughly fractured that in many places it is actually split or can readily be split into thin pieces as thin as or thinner than slate. This cleavage permits the entrance of water near the surface, with the result that at the top the rock has become badly disintegrated, making a weak and slippery material." The schistosity planes constitute the slope of the canyon on the left bank, where there are "slippery sliding surfaces with small adherence, extremely favorable to the formation of large slides when undercut." On the right bank the edges of the schist layers are exposed, permitting a greater penetration of water but not so great a tendency to slide as on the left bank. The schist showed compressive strengths (six samples) from 3,600 to 11,000 lb./in.^{2}, which was not lessened by soaking for 24 hours.

The red conglomerate on the west slope lacks uniformity, and though "when dry it appears to have the character of stone, when wet it disintegrates rapidly to a slippery, friable mass or even loses all shape in a few

minutes. A piece immersed in water gives out bubbles of air practically until the time of collapse, showing that it is thoroughly porous, the absorption amounting to 1.5 per cent of its weight before disintegrating. When wet *in situ* the surface is promptly changed to a slippery substance as little able to keep anything standing upon it from sliding as can well be imagined. Three samples of this were tested dry; they broke at pressures of from 600 to 1,900 lb./in.². Two samples were soaked in water for ten minutes and broke at 210 and 845 lb./in.² respectively. Two other samples, picked from points in the west hillside near the points from where the other samples were taken, were soaked in water for 24 hours and disintegrated into fine particles and sand grains and therefore could not be tested. A large number of additional samples were taken from various points in the west hillside under the dam base and submerged in water and all disintegrated within a few hours."

The cementing material in the red conglomerate consists of clay, iron oxides and amorphous silica. The clay and silica soften in the presence of water, while the iron oxides dissolve. The conglomerate readily absorbs water to the extent of above 1.5 per cent of its weight. The percolation of the water into the conglomerate would soften the cementing material and would dissolve the gypsum which filled some of the fissures in the formation. The effect of softening or removing the binder would be to reduce at once the crushing strength of the mass and its ability to sustain the weight of the dam, and the effect of dissolving the slowly soluble gypsum which filled some of the fissures would be to open up these fissures and loosen the masses of material that are between them. It would also permit freer movement of water into and through the rock. If the water passing through these fissures carried with it gypsum in solution, the water would remain clear and would carry no evidence that it was washing away part of the foundation of the dam. As the leakage increased it might carry with it particles of the conglomerate on either side of the fissures, but in many cases this material contained so little clay or clay-like material that the water would be reasonably clear on emerging from under the dam on the downstream side.

Inspection of the dam by Messrs. Mulholland and Van Norman on March 12 disclosed nothing abnormal or that would cause apprehension. Mr. Mulholland was positive that all seepage streams were clear where they emerged below the dam, although later earth from excavation work near by discolored these streams. The seepage had increased from 7.6 miner's inches (0.16 sec.-ft.) on Feb. 29 to an estimated quantity of 50 miner's inches (1 sec.-ft.) on March 12.

RATE OF FLOW FROM RESERVOIR CALCULATED FROM RECORDING GAGE FIGURES

Gage Time	Least Estimated Flow, Sec.-Ft.	Greatest Estimated Flow, Sec.-Ft.
12:00 midnight	300	1,200
12:20 a.m.	2,000	4,000
12:30 a.m.	10,000	25,000
12:35 a.m.	100,000	150,000
12:40 a.m.	400,000	500,000
12:50 a.m.	600,000	700,000

When the breakup of the dam occurred,

the manner of failure apparently was that the first leak, however started, began under the concrete at that part of the dam which stood on the red conglomerate; this leak increased in volume as it scoured away the foundation material already greatly softened by infiltrated water from the reservoir, which removed the support of the dam at this point and, since no arch action could occur by reason of the yielding conglomerate abutment, made failure of the dam inevitable. The water passing under the dam ran directly down the hillside toward the steep slope of the east bank, causing it to be undermined and to slide. It was such a slide of the bank which early produced the break in the transmission line of the Southern California Edison Company, and not the breaking of the dam itself.

This slide was followed by others, causing the undermining of the east end of the dam. The sequence of failure is uncertain. With the undermining of the dam on both sides section after section of the dam failed, leaving only the central section in place.

Although the recording gage was left with its record intact on the portion of the dam that remained standing, the very small time scale of the record (0.1 in. per hour) makes it difficult to calculate accurately the rate of discharge. Making the best possible approximations the flow was calculated to be as shown in the accompanying table.

Spiral Incline Serves Hospital in Place of Elevators

Uncertainty of Elevator Operation in Chinese City Leads to Use of Reinforced-Concrete Spiral Runway and Stairs

By F. V. Haugsted
Copenhagen, Denmark

AN UNUSUAL piece of construction has recently been completed in a hospital in Mukden, China, involving a spiral runway and staircase of reinforced concrete built around a column in the entrance lobby. The use of elevators was considered to be undesirable, since few had been installed in the city and adequate service and repair parts would therefore be difficult to obtain.

From a desire to have a series of inclines connecting the various floors, the design in Fig. 1 was worked out, providing a spiral runway for wheel-chairs and in addition a spiral staircase close to the column, thus utilizing the space that was too steep for wheeling. The over-all diameter of the construction is 24 ft., providing a 5-ft. width for the driveway, which rises 1:8.15 at the outer

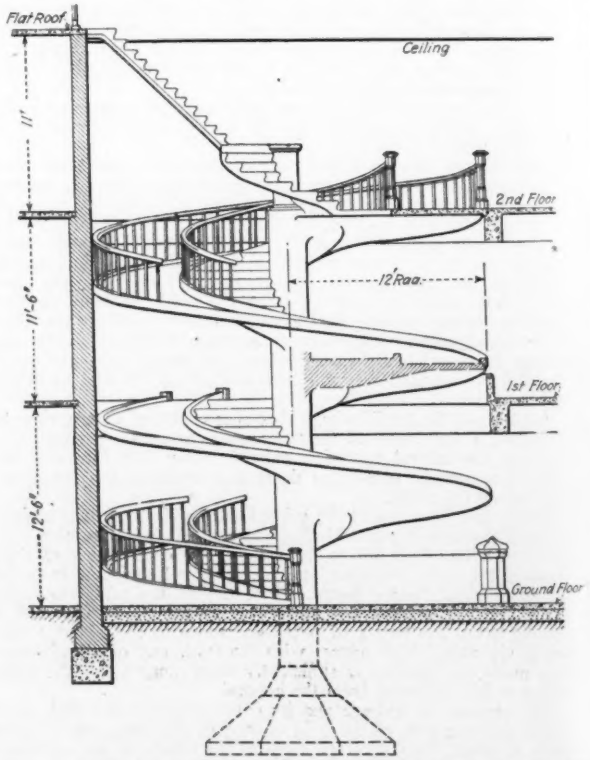


FIG. 1—SPIRAL INCLINE AND STAIRCASE USED IN PLACE OF ELEVATORS IN CHINESE HOSPITAL

circumference and 1:4.65 at the inner circumference. The rise of the path is continuous except at the landings, where sector-shaped platforms are provided.

The 28-in. diameter column is reinforced by eight $\frac{7}{8}$-in. vertical rods and by $\frac{3}{8}$-in. hoops spaced 12 in. The spiral slab is reinforced in the top by $\frac{1}{2}$-in. rods in two directions at right angles, spaced 6 in. apart inside the 18-ft. diameter of the spiral and 9 in. apart outside of this. A radial arrangement would have caused an unsatisfactory crowding of reinforcement at the center column. However, at the points where the pathway is supported by adjoining walls or floors the necessary additional $\frac{1}{2}$-in. rods are arranged radially in the bottom of the slab.

In designing the foundation footing and center column a certain amount of eccentricity of load was considered. The central column was dimensioned with the assumed eccentric loads producing combined compression and bending stresses. the extra stiffening effect which the path lends to the column not being taken into consideration, although account was taken of the fact that the

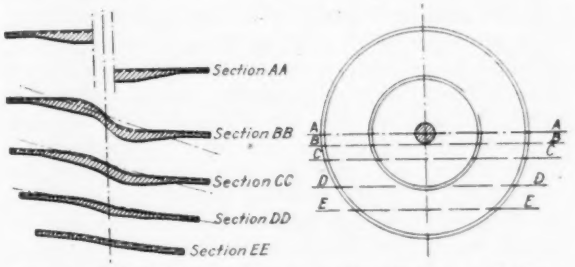


FIG. 2—SECTIONS OF SPIRAL CONSIDERED AS CANTILEVERS AS BASIS FOR DESIGN

column has lateral support where the path intersects walls and floors.

The spiral path was calculated as a series of cantilever beams, a number of parallel sections, *AA*, *BB*, etc. (Fig. 2), being considered, the assumption being in each case that the segment determined by the section was loaded with the total live and dead load. For each of the segments the section modulus was computed and the amount of concrete and reinforcement for each segment determined to make it sufficiently strong to resist the bending moment.

Construction Difficulties—While a reliable portland cement was used, the stone and sand were by no means

FIG. 3—A METHOD OF PLACING CONCRETE IN CHINA All concrete was mixed on the ground and carried to site of placement in the baskets shown.

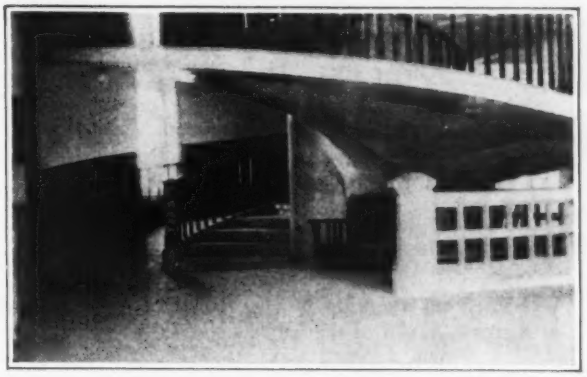


FIG. 4—ENTRANCE TO THE SPIRAL INCLINE IN THE LOBBY OF THE HOSPITAL

of first-rate quality, and even after thorough washing were not free from clay. Concrete mixing was done by hand, and practically all concrete placement was done in temperatures between 82 and 95 deg. F. In Fig. 4 is shown the method of placing the concrete, carried from the ground level in baskets. The basket is one of the universal tools used by the Chinese workman, being used to transport practically everything on the building site.

Reinforced concrete is a comparatively recent material for the Chinese contractor in places like Mukden, and the entire responsibility as to the quality of work is thrown upon the inspecting engineer. On the other hand, the Chinese building carpenter deserves great credit for his skill, considering that he uses nothing but hand tools and that the timber placed at his disposal is in the shape of the bare logs. Such was the case in building the rather intricate forms for the concrete work required by this spiral staircase.

Plan for Railroad Construction in Oregon Again Modified

The Interstate Commerce Commission has again modified the plans for new railroad construction in central Oregon. A previous order of May, 1926 (*Engineering News-Record*, June 24, 1926, p. 1014), authorizing the construction of a new line by the Oregon Trunk Line from Bend, Ore., to Klamath Falls has been revoked in a decision handed down Jan. 24, 1928.

The Great Northern Railway Company is now authorized to proceed with its proposal to purchase an undivided three-fourths interest in the logging road of the Shelvin-Hixon Company, now extending 25 miles to the south from Bend. A new line will be built by the Great Northern from the present terminus of the Shelvin-Hixon line to a point on the Natron cutoff of the Southern Pacific at or near Chemult, Ore. The Great Northern is to have trackage rights from this point to Klamath Falls over the Southern Pacific.

At the same time the commission authorized the use of the tracks of the Deschutes Railroad Company and Oregon Trunk Line between Wishram, Wash., and Bend, Ore., by the Great Northern. With the new construction south of Bend, this gives the Great Northern direct access to southern Oregon from the main line at Wishram.

The commission also authorized the Southern Pacific and the Great Northern jointly to acquire control of the Oregon, California & Eastern Railway Company by purchase of capital.

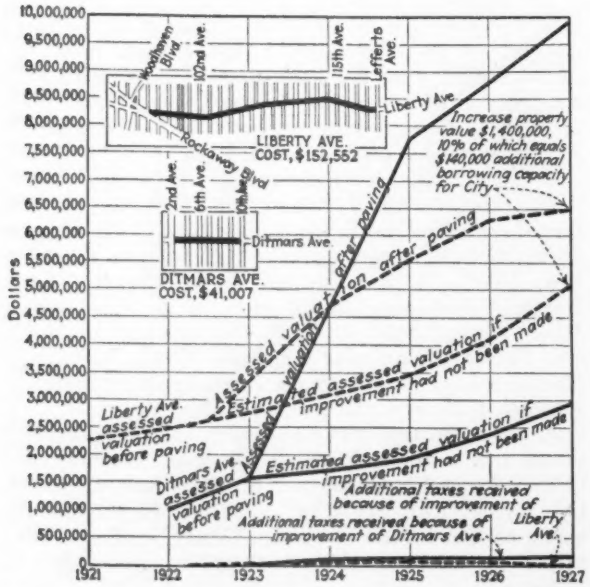
Public Improvements as a Source of City Revenue

Illustrated by Increased Assessed Valuations, Tax Yield and Borrowing Power Due to Street Paving

By Charles V. Powell
Engineer in Charge Topographical Bureau of Queens Borough, New York City

THE recent adoption by the Board of Estimate and Apportionment of the City of New York of a record-breaking budget appropriation of $512,528,831 for the year 1928 has evoked the question of how this revenue is to be raised without undue hardships on property owners. Although most of the increases over the 1927 budget are mandatory and 50 per cent of the entire budget, or over $260,000,000, is represented by debt service and expenses of the Police Department and the Board of Education, while less than 5 per cent is required to finance the running expenses of the five borough governments, nevertheless it is due largely to the activities of the latter that the city is able to increase its budget year after year without materially increasing its tax rate.

These activities—which increase the wealth of the boroughs and therefore the income to be derived from taxation—consist of improvements, such as sewers and highways, which are paid for chiefly, and in the case of sewers entirely, by local assessments upon the property benefited. While the property owners have the option of distributing their assessment payments over a period of ten years, during which time the city is required to finance the cost, in many instances the entire cost of the improvements is returned to the city within two or three years after the completion of the work through the increased assessable values thereby resulting. This is in addition to the assessments collected meanwhile from property owners.



HOW PAVING BUSINESS STREETS INCREASES PROPERTY VALUES AND CITY BORROWING POWER

Ditmars and Liberty Avenues are in the Borough of Queens, New York City. The estimated assessed valuations if these avenues had not been paved are based on average assessments for the whole borough. Borrowing power is limited to 10 per cent of assessed valuation.

That this increase in the assessable value of property is not caused by inflation is shown by the record of real estate transactions in which the actual prices received for property within the areas of such improvements are greater, and in some cases several times greater, than the assessed values upon which the taxes are based.

At the request of Borough President Maurice E. Connolly the writer has made studies of the effects of various highway improvements in increasing property valuations in Queens Borough. Some of the results have been astounding in the rapid increases shown in valuations. In the accompanying graph two specific instances, taken at random, show the rapid increases in valuation which have resulted from the comparatively moderate sums which have been spent on the improvements.

Valuations Increase After Paving

In the case of Ditmars Ave., the cost of the paving was $41,006. Within one year after this pavement was completed, the assessed valuation of the adjacent property rose from $1,600,000 to $4,700,000, thereby increasing the borrowing capacity of the city by $310,000, on the basis of its borrowing capacity being 10 per cent of the value of the assessable property within its limits; or the increase in its borrowing capacity was more than seven times the cost of the improvement. In two years the assessed valuation of the same property rose to $7,750,000 and the city's borrowing power was increased by $615,000, or fifteen times the cost of the improvement. Of this $615,000, the sum of $32,000 represents the amount due to the average secular increase of property valuations in the entire borough, leaving $583,000 additional borrowing capacity to be credited to the improvement. At the close of the third year this additional borrowing capacity had been increased to $640,000 and at the end of the fourth year to $690,000. Besides this increase in borrowing capacity, the city received in additional taxes $60,090 at the end of the first year, $118,349 at the end of the second, $129,950 at the close of the third and $140,070 at the end of the fourth year from the completion of the improvement costing $41,006. Furthermore, the city received from special assessments, due to the paving work, a total of $20,503 in four years.

On Liberty Ave., the cost of the paving was $152,551; and the assessed valuation of the adjacent property rose from $2,650,000 to $3,350,000 at the end of six months, or the beginning of 1923, to $4,700,000 by 1924, $5,550,000 by 1925, $6,300,000 by 1926 and $6,500,000 by 1927. Meanwhile the borrowing capacity of the city had been increased $140,000 above the average of the entire borough, due to the improvement; and the sum of the additional taxes collected between 1924 and 1927 inclusive had amounted to $149,205 over and above the $122,041 paid in installments by property owners to liquidate the cost of construction.

Increase Not Due to Inflation

These increases in wealth are not due to inflation, but represent the transmutation of latent into current values, that can be cashed in by any owner willing to sell his property. They explain, at least in part, why the City of New York is able to increase its budget without augmenting its tax rate and to include in its 1928 budget an entirely new item of $13,000,000 toward the cost of subways, and how it can face with equanimity the prospect of continuing to finance in the future a part of its subway program on a "pay-as-you-go" basis.

Administering Farm Lands in Flood-Retarding Basins

Miami Conservancy District Bought 30,000 Acres Behind Detention Dams—Buildings Relocated —Operating the Farms

By C. H. Eiffert
Chief Engineer, Miami Conservancy District, Dayton, Ohio

A FLOOD-CONTROL project like that of the Miami Conservancy District presents many problems. Closely interwoven with the engineering work of the district is the problem of the farm lands in the five retarding basins or reservoirs.

The principal point of interest lies in the fact that these are dry reservoirs, as contrasted with the usual type of power, irrigation or water-supply reservoirs. No water is stored in them except temporarily in times of flood. There are no gates; the outlets are permanently open. Control of floods is thus entirely automatic. Practically all damaging floods occur during the winter

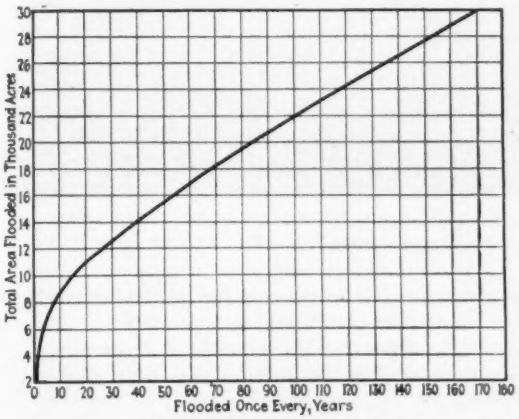


FIG. 1—RELATION BETWEEN AREA AND FREQUENCY OF FLOODING
Total curve for five detention reservoirs.

or early spring and therefore do not affect agricultural operations except that they leave a deposit of silt, which enriches the soil. Most of the lands are fertile bottom lands, the best farm lands in the entire river valley. It was therefore apparent from the beginning of the project that to keep them producing would mean a considerable saving to the district.

The total area in affected farms back of the dams is 55,000 acres. It was necessary for the district to purchase outright 30,000 acres and to secure flood easements on the remaining 25,000 acres. The total area actually subject to overflow in a maximum flood—that is, a flood 40 per cent greater than that of 1913—is 35,600 acres. A flood equal to that of 1913 would overflow 29,700 acres. The estimated frequency of flooding is shown on the accompanying diagram, Fig. 1. Only small parts of some of the affected farms are subject to overflow, and these farms were easily taken care of by flood easements. In a few cases agreements were made to pay damages as flooding occurs. So far it has not been necessary to pay any such damages.

The 30,000 acres of land purchased outright cost an average price of about $150 per acre. Of these 30,000 acres, 16,000 acres have now been resold at an average of approximately 80 per cent of the purchase price. Most of the land sold was purchased by local farmers. The lands remaining in possession of the district are divided into farms of from 75 to 1,400 acres, and as much as 6,000 acres could be sold in one body. About 3,500 acres will be held by the district as permanent reserves connected with the dams.

Relocating Farm Buildings—Many of the buildings on the farms purchased in the lower part of the basins were located at such low elevations that they had to be moved to higher ground. This meant also a rearrangement of the farm lines and field lines to adjust them to the new farmstead locations. In the rearrangement and readjustment of the lands several ends were kept in view. One was that the farms must be so handled as to yield the maximum return in crops not only after the readjustment but during the transformation period as well. Another was that the farms as an investment should suffer no injury and if possible be improved. A third was that the farms should be resold to private owners as soon as possible, the district retaining a flood easement.

Altogether no less than 100 farmsteads were removed on account of being below the allowable limit. Twenty-four new sets of buildings were constructed by the district. Some of the individual buildings were moved from old sites to new locations and were there remodeled and repaired, whereas in other cases entirely new buildings were constructed. Three sets of buildings were relocated by the purchasers of the land, making a total of 27 new farmsteads.

Fifteen farmsteads remained in their original locations but were added to, improved and enlarged in order to serve larger areas of land than before. Nine of these were remodeled by the district and the six others by the purchasers of the farms.

The regulations for buildings in the retarding basins provide that no buildings shall be permitted to remain that are on ground lower than 10 ft. below the spillway, and no new buildings shall be constructed on ground lower than 5 ft. below spillway level. There naturally

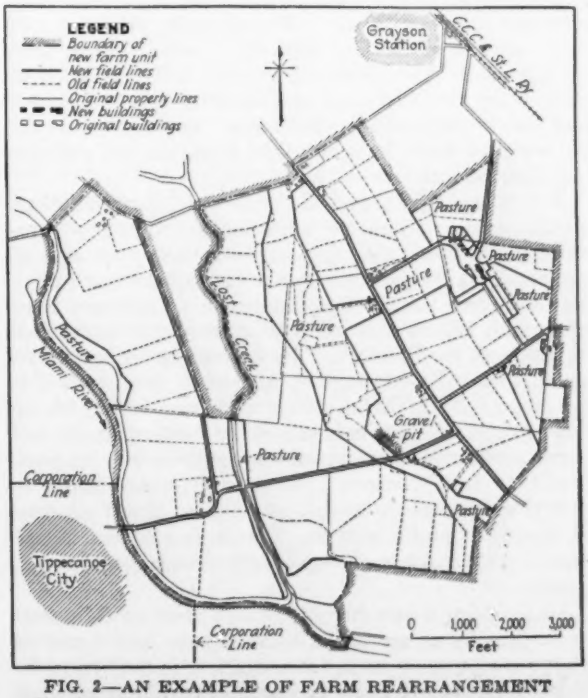


FIG. 2—AN EXAMPLE OF FARM REARRANGEMENT
Twelve old farms combined into one 1,400-acre farm, because only one site was available for buildings.

arose conditions to which these rules could not be strictly applied. In a number of cases it was considered necessary to remove buildings that were less than 10 ft. below spillway, especially if they were one-story dwellings with no higher ground near. In a number of other cases requests were made for buildings to remain that were more than 10 ft. below spillway. In such cases a special survey and study was made of each one. In some cases, where high ground existed near by and the route of escape to such ground was easily accessible, they were permitted to remain. All deeds given by the district provide that no new buildings shall be erected lower than 5 ft. below spillway without the written consent of the district. Periodic inspections are made to see that this regulation is complied with. In several cases this ruling was ignored or overlooked and buildings were erected below the permitted level without the district's knowledge or consent. These buildings were removed by the owners at their expense.

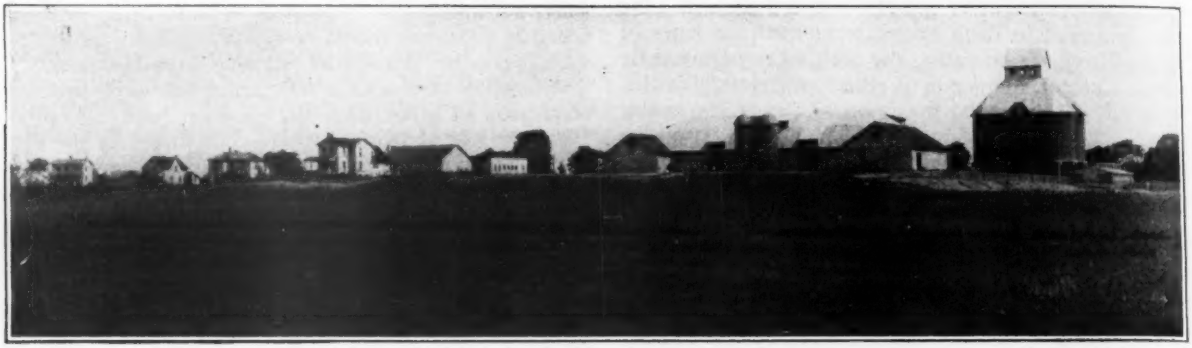


FIG. 3—MODERN FARMSTEAD OF 1,400-ACRE UNIT
Granary and hog-house on right, dwellings on left.

A Replanned Farm—Fig. 2 is a map of the largest farm unit on the project. It contains 1,400 acres and is composed of twelve farms that were purchased from as many separate owners. The necessity for a unit of this size lies in the fact that there are only two sites on the entire tract high enough for buildings, both of these being close together near the east side of the tract and both on the same original farm. The map shows the old and new farm lines and field lines, the old and new buildings and the roads.

The photograph shows the new set of farm buildings on this unit. There are four modern dwellings provided with hard and soft running water, plumbing, electric lights and power. The residences are much more desirable than the average farmhouse. There are large barns with 15,000 sq.ft. of floor space and an equal area of basement for livestock. The basements are connected by an inclosed passage with a modern granary which has space for 32,000 bu. of ear corn and 13,000 bu. of small grain. All grain is put in by modern dump and power elevator and withdrawn by gravity into wagons, trucks or feed carriers. An up-to-date hog-house is located close enough to this granary so that feed may be spouted directly into it. There is also machinery storage space and a workshop equipped with forge, power grinder, etc.

An unfailing water supply is taken from an 8-in. well 50 ft. deep by an automatic electric pump and stored in a large compression tank, from which it is distributed to all buildings. Soft water is stored in two large cisterns and is supplied from the large barn roof area. This is handled by the same type of pump and stored in a smaller compression tank. Sewage is taken care of by a septic tank. To make possible the use of electric power it was necessary to build 2 miles of transmission line. This was paid for partly by the district and partly by the power company. The current is carried to the buildings at 6,600 volts and stepped down to 220 for power and 110 for lighting purposes.

Increased Efficiency—It was possible to supply these modern conveniences and labor-saving devices on a large single unit, although their cost of installation and maintenance on the twelve original farms would have been prohibitive.

There is economy also in the remodeling of field lines. The average size of cultivated fields on the original farms was 11.2 acres, whereas on the new unit it is 85 acres. Less labor per acre is required for all field operations as well as less labor per bushel and per head of livestock.

Although ground high enough for buildings was scarce, the overflow feature on this particular unit is negligible, as the lowest field will probably not have backwater oftener than once in ten years. Silt deposit can therefore not be depended upon for fertilization. The soil, however, is of superior quality and is being constantly improved by proper fertilization treatment and rotation of crops.

While the above described is the largest unit in the district, the improvement of buildings, fields and method of handling is typical. By repair and remodeling of buildings, relocation of field lines, proper crop rotation and fertilization, drainage of wet spots, building of fences, etc., all of the farms have been brought into much better condition than they were when first taken over.

As stated, all of the lands have been kept in cultivation since their ownership by the district. A few farms are rented for cash, but most of them on a share basis, the district furnishing one-half of the seed and fertilizer and receiving one-half of the crops raised. The district's shares of crops are marketed by the tenants at the district's instructions. The renting of farms, the sale of crops and the district's interests in general are taken care of by two farm supervisors, who have a practical knowledge of farming and sufficient business ability for proper handling of the district's affairs in connection therewith.

Land Permanently Reserved—Mention has been made of the permanent reserves. These lie immediately above and blow the dams and consist largely of steep timbered hillsides, with a small amount of tillable bottom land. Part of this land is used in each case for the care-

taker's residence and grounds. The tillable land not so used is rented. The use of some of these areas for recreational purposes by the public has been permitted where such use does not interfere with the operation of the flood-control system. Many residents of the valley as well as tourists take advantage of the opportunity to use these grounds, and campers may be found there at all times during the summer. The district's patrolmen keep the grounds in order. Two tracts have been leased to the state for game preserves.

The reserves as well as other timbered tracts owned by the district have been carefully studied by an experienced forester and are being handled to the best advantage. Mature trees are being removed in some cases, barren hillsides are being replanted with desirable varieties, and the young growth is being protected.

The entire farm land program since the beginning of the project has been under the direct supervision of S. Graham Smith, farm manager.

Fourth Sewage-Works Completed at Madison, Wis.

Two Plants Abandoned and Third Will Be—New Plant Has Grease Extractor, Tanks, Filters and Chlorinator

EXCEPTIONALLY rapid growth of population at Madison, Wis., was the main reason for building the Nine Springs sewage-works, as also for building the old plant now in use. Engineers' reports in 1912 estimated a sewage flow of 6 m.g.d. for 1937 and 7 m.g.d. for 1941, but these amounts were reached in 1922 and 1927 respectively. Curves of growth prepared by the city engineer now indicate 13 m.g.d. for 1941.

The new plant is the fourth one built by Madison. Two of them were abandoned successively and the third will be abandoned when the new works are in full operation. The first plant, using a patented chemical precipitation system and sand filters, was put into use in June, 1899, and abandoned as unsatisfactory in January, 1901, In this latter year a new plant was put into operation, designed by F. E. Turneaure, dean of the engineering college of the University of Wisconsin and at that time city engineer. It included septic tanks and cinder filter beds, with a capacity of 1.5 m.g.d. on a twelve-hour settling period, but in 1914 it was treating 3.5 m.g.d.

In view of this overloading, a larger plant was designed in 1912 by E. E. Parker, city engineer, with John W. Alvord as consulting engineer. This was of the separate sludge digestion type, with a rated capacity of 5 m.g.d., or 7½ m.g.d. for short periods of overload. The sewage tanks were put into service in 1914, but the sprinkling filters were not completed until 1916. These works (*Engineering News-Record*, Sept. 11, 1919, p. 510) were expected to serve without overloading until 1930 or later. In 1920, however, the plant was being worked beyond its capacity, and in 1923 the city council adopted the recommendations of the city engineer, E. E. Parker, for a plant of 5 m.g.d. to include Imhoff tanks with sprinkling filters and secondary sedimentation. Its capacity was based on the plan of continuing the old plant to serve the east end of the city, but in 1924 it was decided to provide for a future enlarged capacity of 10 m.g.d. and ultimate abandonment of the old plant. The present new plant as built has a capacity of 5 m.g.d., but provision is made for duplication and a future capacity of 10 m.g.d.

The Nine Springs plant, located south of the city, comprises: (1) Grit chamber and grease extractor, (2) Imhoff tanks, (3) dosing chambers, (4) sprinkling filters, (5) chlorination, (6) secondary sedimentation, (7) sludge beds, (8) sludge-pumping station, also office and chemical laboratory. Sewage comes to the plant through a 30-in. cast-iron force main 3½ miles long, from the Greenbush pumping station. The plant effluent flows through a concrete outfall sewer 4½x3 ft. and 1,500 ft. long to a drainage ditch discharging into the Yahara River just above Lake Waubesa. Later this sewer will be extended 6,800 ft. to the river.

Although the city has the separate sewer system, the sewers receive considerable grit from street inlets and roof drains, so that a grit chamber is needed to keep this material out of the sewage-works. This chamber is so designed that grease and floating scum can be removed by raising the head by means of a butterfly valve, partly closing the outlet. Two hopper-bottom channels with baffles give a detention period of 5¾ minutes. Grit removed with a clamshell bucket will be placed on an 18-in. coke drying bed with tile underdrains. Six two-hopper Imhoff tanks, designed for reversible flow, give a capacity of 5 m.g.d. for raw sewage, with a detention period of 2½ hours. The sludge storage is 2.75 cu.ft. per capita for 40,000 population; the gas vent area is 24 per cent of the total tank area. Skimming compartments at one end of each gas vent permit the scum to be flushed into the sludge channel beneath the influent and effluent channels. To equalize flow to the tanks, each tank has an inlet valve and venturi meter, so that the flow can be checked at various heads.

A diversion manhole with weir chamber insures uniform flow to the twin dosing tanks, each of which has a capacity of 26,000 gal., or fifteen-minute flow, discharging through a 24-in. automatic siphon in 5½ minutes to the sprinkling filters. The two filters, 272x178 ft. each, are 10 ft. deep, filled with crushed dolomite limestone. At the bottom is a 12-in. layer of 3- to 5-in. stone, then 7 ft. of 1½- to 3-in. stone, and 2 ft. of ½- to 1-in. The maximum effective head on the nozzles is 8.6 ft. There are 286 nozzles on each filter, spaced 14 ft. c. to c. The principal distribution main supplies a series of laterals, each of which carries thirteen nozzles and is provided with a shutoff valve and a blowoff valve which discharges into the effluent channels in the pipe gallery.

A chlorination unit for use in summer or during the bathing season will be placed between the filters and the clarifiers, which give a one-hour detention period. The final treatment is the secondary sedimentation in two Dorr clarifiers, from which the sludge will be pumped to the sludge compartments of the Imhoff tanks. Sludge beds having 2½ ft. of gravel covered with 6 in. of sand and having tile underdrains have an area of one acre, or 1 sq.ft. per capita. Concrete planks and posts divide them into 20-ft. sections, each with industrial track for sludge removal.

Plans and specifications were prepared by E. E. Parker, city engineer. W. A. Peirce was in charge of construction, Louis S. Davis was office engineer and C. W. Gates, inspector. Pearse, Greeley & Hansen, Chicago, were consulting engineers as to the clarification devices. The contractors, all local firms, were as follows: Nicholas Quinn, grading and filters; John R. Cullinane, force main; George Nelson, tanks and buildings.

Repairing Construction Defects in a Concrete Building

Open Joints in Beams, Cavities in Columns and Rotten Concrete Repaired by Guniting — Shallow Eccentric Caissons Rebuilt Under Ingenious Shoring System

By Grayson Gill
Engineer, Herbert M. Greene Company, Dallas, Tex.

A Remarkable Disclosure of construction faults in reinforced-concrete building is made in this article. Adequacy of structural design is not in question. The faults lie in the way the construction work was performed and passed inspection. About every possible error in placing concrete and reinforcement appears to have been committed. Even essential structural parts were wrongly located. Common foundation soil requirements had been overlooked. Originally designed to carry eleven more stories, it is to be noted that these faults were not disclosed until examination of the lower stories and foundations was undertaken preparatory to the construction of the addition. The building had in the meantime been in constant use and had served every purpose to which it had been put.
—Editor.

EXTENSIVE alterations and the erection of fourteen additional stories were recently undertaken on a four-story reinforced-concrete garage building which was completed in the fall of 1925. The architect and the general contractor engaged to do the new work had no connection with the design or erection of the original structure, and in adapting that structure to the requirements of the new plan, which varied materially from those provided for originally, numerous defects were brought to light requiring extensive repair work before the contemplated new work could proceed. All these defects arose from bad practice in field work, the structural design by a competent engineer being above criticism, and they illustrate how a careful design can be completely nullified by a contractor in the absence of adequate supervision.

The design of the original four-story structure provided for eleven additional stories, but a test of the rock taken from a test pit dug beside one of the caissons indicated that the caissons as designed were sufficient to carry the fourteen additional stories of the new design.

Beam Defects—The floors of the old building were reinforced-concrete joists and beams. A visual inspection showed that construction joints with few exceptions were located at points of high shear in beams and joists near supports and that at these joints in many cases the concrete was in bad condition, as illustrated by Fig. 1.

FIG. 1—UNDER VIEW OF FLOOR SHOWING TYPE OF DEFECTS

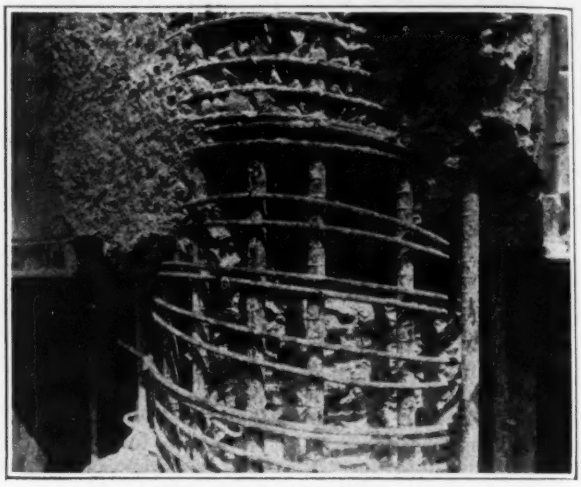


FIG. 2—CAVITY IN COLUMN UNDER SPANDREL BEAM

Of special interest was a group of construction joints which very closely approximated the planes of failure by diagonal tension. These had opened up enough to allow the insertion of a knife blade and when a passenger car was run over the slab the movement of the joint perceptibly pinched the fingertips held against it. Two years of service had not resulted in a complete failure at these weak points, but when air hammers were put into operation cutting out a section of floor in nearby panels for new elevator shafts, the vibration of the structure caused some of this defective concrete to come loose and fall off.

Defective beams and slabs were repaired by shoring where necessary, removing all loose defective concrete, cutting out all weakened construction joints at points of high shear and rebuilding to the original dimensions with gunite. In two instances 12x24-in. beams were found from which the wooden bulkhead forming the construction joint had not been removed, and these joints were within 3½ ft. of the supports! Another common defect in heavily reinforced beams was the crowding together of the reinforcing bars in the bottom of the beam and so close to the forms that no coarse aggregate could pass through or around them. The result was a thin hollow shell of cement and sand on the bottom of the beam, with consequent bond failure, marked deflection and bad cracks. These beams were reinforced with additional steel and gunite.

Column Defects—Where very wet or very dry undersanded concrete was poured in wall columns, the bottom

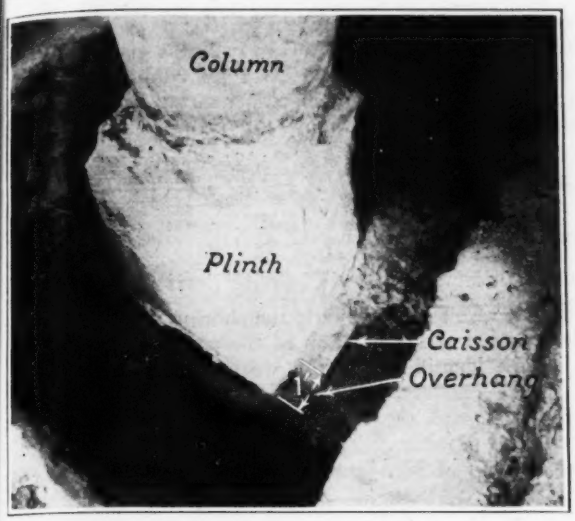


FIG. 3—COLUMN PEDESTAL ECCENTRIC ON FOUNDATION CAISSON

bars of the spandrel beams in several instances held all the coarse aggregate like a screen, leaving large pockets the equivalent of a foot or more square and 3 to 6 in. deep in the core of the column, as illustrated by Fig. 2. These were located by sounding at suspicious points where extensive pointing up had been done and were cleaned out like a decayed tooth and filled with gunite.

Foundation Defects—A basement extended under a part of the building, and one column supporting the new structure was located in this basement. A test pit was dug by the architects beside the caisson supporting this column to verify the depth and the character of rock on which the caissons rested. The depth was found to be as shown on the drawings for the old building and the rock was satisfactory for the loads to be carried on the old foundations as also shown on the drawings. When construction work was started, however, and the ground-floor slab was removed from around the columns to place gunite column jackets directly on top of the caisson, it was found that the caissons were not built according to the plans, but that the top of the square caisson terminated approximately 3½ ft. below the floor line and a batter-faced plinth extended from the top of the caisson to the floor.

The first caisson uncovered was found to have this plinth overhanging the edge of the caisson so far that the side of the round column was 3½ in. outside the face of the caisson, as shown in Figs. 3 and 4, making an eccentricity of 20½ in. No dowels tied the plinth to the caisson, and when the latter was cut away, a smooth laitance-covered surface was left, showing that there had been no bond between the two. A test pit dug beside this caisson revealed that it did not extend to bedrock as shown on drawings and as certified by the builder, but stopped on a soft white chalk formation inadequate for the loads it was designed to carry. Test pits were subsequently dug beside the other caissons, and in no instance was a safe foundation found to have been built. A total of fifteen columns were shored up, the old concrete caissons removed and new caissons poured underneath them.

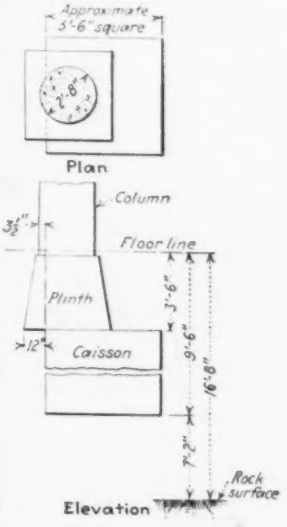


FIG. 4—SKETCH OF FOOTING SHOWN IN FIG. 3

It was clearly demonstrated on this job that the least satisfactory bearing is found near an impervious stratum, in this case bedrock. In one instance the foundation had been carried to within 2 ft. of rock, and when the bottom of this caisson was exposed, soft material flowed out beneath it into the test pit.

The evil effects of dropping concrete into water were also well illustrated in several of the old caissons. Laitance as much as 10 in. thick, containing clay balls and débris, was found at the tops of caissons directly beneath the plinth. Below the laitance was a section of soft but tough oversanded concrete, while the bottom 3 to 3½ ft. consisted of coarse sand and gravel containing practically no cement binder. When this latter was shoveled out of the pit, it looked like a pile of pit run gravel.

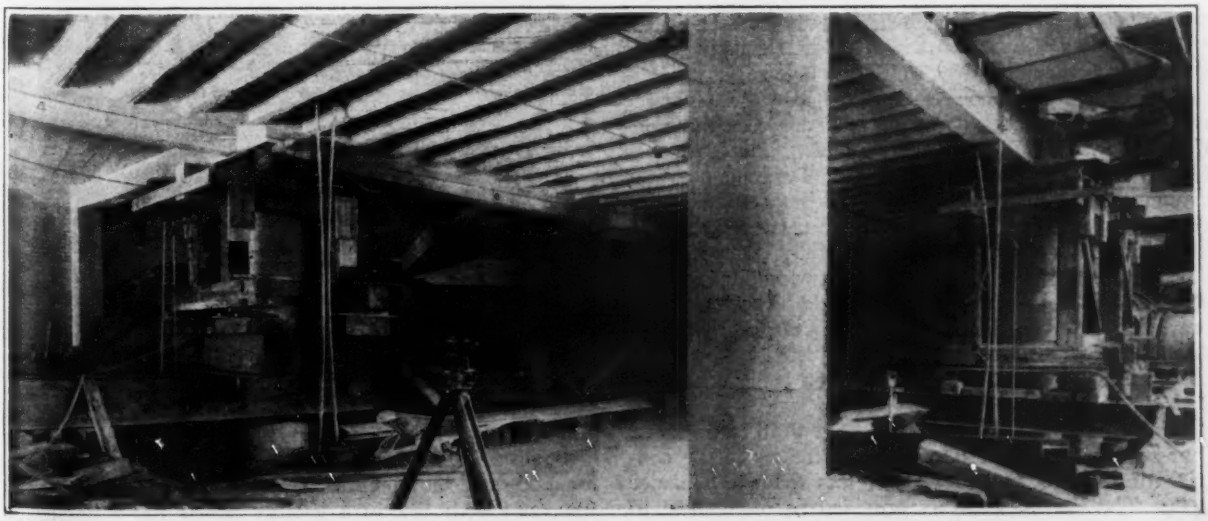


FIG. 5—VIEW OF SHORING SYSTEM FOR REPLACING FOUNDATION CAISSONS

Shoring Methods—The view, Fig. 5, and the sketch, Fig. 6, illustrate the method adopted for shoring up the columns while defective caissons were being removed and rebuilt. A gunite shell 3½ to 4½ in. thick had already been placed around all the columns except those in the first story, so by cutting into the fireproofing on each side of the first-story columns just below the octagonal cap sufficient bearing and shear area were secured to develop the entire dead load carried by the column. The building had been temporar ly vacated by the tenant and no live load was involved. These dead loads were computed and the shoring details designed accordingly.

In order that no strains might be put on the relatively rigid members of the concrete frame incidental to this

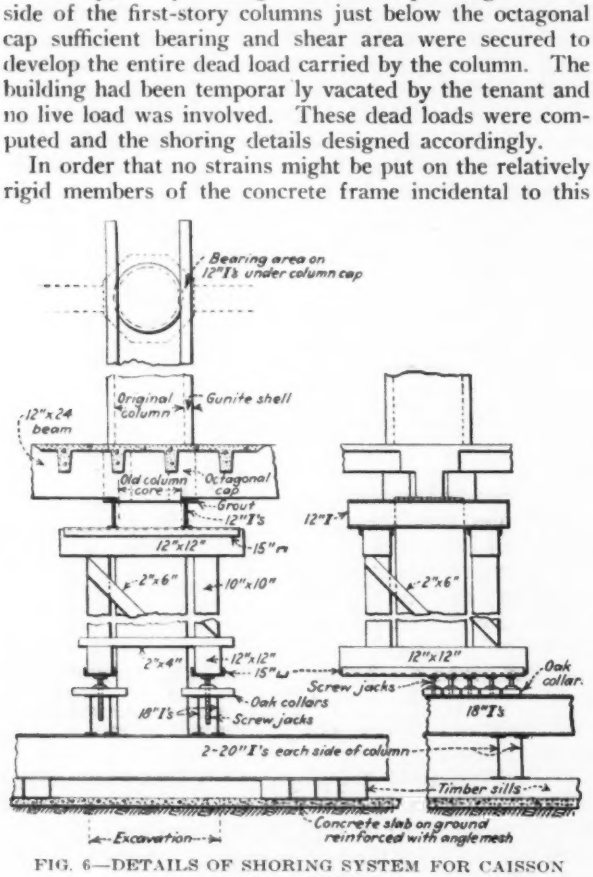


FIG. 6—DETAILS OF SHORING SYSTEM FOR CAISSON REPLACEMENT

operation, an adaptation of the simple strain gage devised by F. F. Sinks, of Seattle, used in connection with the repairs to the Holland Building in that city and described in *Engineering News-Record* of Dec. 11, 1924, was used on this work. This gage can be seen in the view, Fig. 5, and consists of two 1x2-in. clear pine strips with a 1x2x12-in. header top and bottom, the strips bowed inward with about 2 in. clear between them at the center. A pointed stick was cut to fit between these uprights a few inches below the center. This gage was fitted in tightly between the floor and ceiling near the column where the bottom would not be affected by the cribbing load. The screw jacks were tightened up until the stick fell out and then were carefully tightened further until a strain was secured that would produce a spread of ⅛ to ¼ in. more on the gage. The high compressive stress on the horizontal timbers generally produced a permanent set, which was adjusted by taking up on the jacks twice a day, this operation being controlled with the gages.

High-Early-Strength Concrete—The dead loads of the four-story structure resulted in low stresses in the concrete of the caissons, in general less than 100 lb. per sq.in. The shoring contract provided for equipment to shore only five columns at a time, so it was desired to release the shoring equipment as soon as possible. Concrete in the proportions of 1:1:2 was specified, using standard portland cement with no admixtures, and a maximum total water of 5 gal. per sack. This gave compressive strengths as follows of 6x12-in. cylinders:

No.	Strength Lb. Per Sq.In. 24 Hr.	72 Hr.
1	810	....
2	988	....
3	906	....
4	..	1,770
5	..	1,525
6	..	1,923
Average	901	1,739

As a result of these tests the shoring was removed 36 hours after the completion of the pouring of each item.

Those identified with this work were the Herbert M. Greene Company, architect; R. O. Jameson, consulting engineer; and the McKenzie Construction Company, general contractor. The work was executed under the supervision of Mr. Jameson and the writer.

Reclamation Bureau Refutes Charges of Exceeding Estimates

To meet charges that the Bureau of Reclamation habitually exceeds its estimates in the construction of dams, Dr. Elwood Mead, head of the bureau, has had a table prepared to show the estimated costs and the actual costs of the more important dams constructed by the bureau. The table follows:

Name	Project	Estimated Cost	Actual Cost
American Falls (c)	Minidoka	$8,500,000	$7,300,000
Arrowrock (c)	Boise	6,250,000	4,496,731
Avalon	Carlsbad	162,000	315,989 (i)
Belle Fourche	Belle Fourche	1,040,416	1,259,515 (k)
Black Canyon	Boise	1,800,000	1,492,305
East Park	Orland	198,000	196,120
Echo	Salt Lake Basin	1,394,590 (d)	1,125,098 (e)
Elephant Butte (c)	Rio Grande	5,600,000	5,004,216
Gibson	Sun River	1,826,195 (d)	1,566,240 (e)
Guernsey	North Platte	1,780,000	1,700,351
Kachess	Yakima	712,000	661,000
Keechelus	Yakima	1,069,000 (f)	1,892,778 (m)
Laguna	Yuma	972,455	1,980,462 (f)
McKay	Umatilla	2,500,000	2,116,828
Pathfinder (c)	North Platte	1,000,000 (h)	1,794,366 (d)
Roosevelt	Salt River	3,750,000 (a)	3,806,277 (b)
Shoshone	Shoshone	1,000,000 (j)	1,439,135
Stony Gorge	Orland	609,524 (d)	518,904 (e)
Tieton (c)	Yakima	4,020,000	3,756,256

(a) 190-ft. dam. (b) 220-ft. dam. (c) Dam and reservoir. (d) Engineer's estimate. (e) Contractor's bid. (f) The surface of the dam was paved with concrete instead of rock as originally intended due to poor quality of rock obtainable. Notwithstanding predictions of geologists, the rock uncovered in the quarries was found unsuitable for such paving, and its use had to be abandoned in favor of concrete. Sluiceways were also paved with concrete for the same reason. There was considerable waste in quarrying, at times 50 per cent, due to poor quality of rock, thereby greatly increasing excavation quantities. Use of sheet piling had to be considerably increased. The river break into Salton Sea increased transportation difficulties by rendering the river unnavigable. There was a large increase in cost due to increase in price of labor and materials. (g) Increase partly due to the building of an additional outlet tunnel, and changes made in north tunnel, both together amounting to $641,000. (h) No detailed estimate found, but early board reports show $1,000,000 allowed for Pathfinder reservoir. (i) Increase due to use of concrete core-wall instead of sheet piling, two new tunnels to increase spillway capacity and an additional spillway of reinforced concrete. These changes cost more than $100,000. (j) No detailed estimate found, but early correspondence gives $1,000,000 as the preliminary estimate. (k) Failure of contractors delayed work two years, and this, together with additional construction of a gravel berm and installation of auxiliary valves, increased the estimated cost. (l) Modified by board report of Dec. 16, 1913, to $1,337,000. (m) Difficulty of obtaining suitable material increased cost by $240,000. Other changes which greatly increased the original estimate were riprapping, inclusion of concrete cutoff wall, changes in tunnel scheme, increased excavation for spillway and heavier concrete lining, additional road construction and clearing and logging reservoir, the latter item alone costing $290,000.

Cannot Change Zoning for Single Property Owner

Three courts of New York State—Supreme, Appellate and Appeals—have ruled that the zoning regulations of New York City cannot be changed by the local Board of Standards and Appeals for the benefit of a single property owner. The decisions uphold the superintendent of buildings in refusing to grant a corporation a permit for a 21-story building at the southwest corner of Park Ave. and 40th St., in a locality zoned for residences, known as the Murray Hill section.

Checking Earth Slips by Drains Paralleling Road

Perforated Metal Pipe in Crushed Stone Drains Demonstrates Efficiency and Indicates Construction Methods

By N. F. Hathway
Maintenance Engineer, Fourth Division, Morgantown, W. Va.

Land Slips cause great damage to roads in the hilly regions of West Virginia which extend back from the Ohio River. The nature of these slides and the various methods employed to check them were described in an article by H. J. Spelman, division engineer, West Virginia State Road Commission, in *Engineering News-Record*, March 17, 1927, p. 434. In the following article the problem of slides and their prevention is discussed further, particularly along the line of remedial drainage. The hazard of slides is a very serious one in certain regions and definite information of ways in which it can be guarded against is of interest. —Editor.

SLIPS directly and indirectly account for approximately one-fifth of the total expenditure for highway maintenance in the fourth division of West Virginia, comprising twelve counties in the mountainous north-

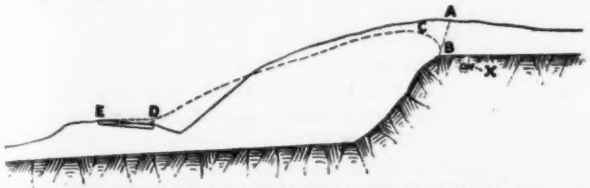


FIG. 1—TYPICAL SIDEHILL SLIP ABOVE ROAD

The first indication is usually a crack along *AB*. The material between *CB* and the road then gets into motion and the surface assumes position *CBDE*, quite often raising the pavement as shown at *DE*. Such a slip can be stopped by collecting water in trench as shown at *X* and leading it around slip in controlled channels.

central portion. Specifically the cost of maintenance due to slips in 1927 was $90,000. The records show that in 1927 slips caused this division to move 77,760 cu.yd. of earth and rock. In addition 5,000 sq.yd. of pavement was replaced and 600 cu.yd. of wall was built. To arrest slips 14,158 lin.ft. of piling was driven and 3,880 lin.ft. of 8-in. perforated pipe was used.

Piling has been resorted to only when the source of water could not be located, and in more than 50 per cent of the places used it has not held the slips for a period of one year. The piles have, however, arrested the slips sufficiently to keep the roads open to traffic.

As nearly all slips are caused by water lubricating a clay formation, usually overlying soapstone, collecting this water above the slip and diverting it through controlled channels will stop the earth movement in the majority of cases. The difficulty is to locate the water at a point above the slip. In only a few cases does it appear on the surface; it is commonly from 6 to 12 ft. below the surface at the point where it is desired to collect it. For this reason an open trench is impracticable. The practice is to place in the bottom of the trench one or more lines of perforated pipe, surrounded with broken stone or slag. The perforated pipe used is corrugated metal pipe with perforations in the lower third. It insures a positive opening at all times, and as the joints are firmly fastened together, it is preferable to ordinary farm tile.

As an example, in Lewis County there was a slip above the road which extended along the road nearly 200 ft. and uphill from the road for about the same distance. The material was in constant motion, had filled the ditch and crowded out on the pavement. This slip required constant attention, and several hundred yards had been removed before it was decided to drain it from above. A ditch was dug nearly parallel to and 200 ft. above the road. It was necessary to go down about 12 ft. to reach the stratum upon which the slip was taking place. Two lines of perforated pipe were placed, surrounded by broken rock. This installation was made more than twelve months ago, and since that time less than 50 cu.yd. of material has been removed from the slip.

Another type of slide quite common is where the upper ditch is porous and the water leaves the surface to cross under the road at one or more points, causing slides on the lower side. The procedure has been to dig a trench in the upper side ditch down to and 8 in. or 1 ft. into the impervious stratum; lay broken stone and perforated tile in the bottom of the trench, and fill the trench to the surface with broken stone. If there is a convenient culvert at an elevation low enough to take water across the road, it is utilized; if not, a new culvert is placed. Often it is necessary to construct a drop inlet where the perforated pipe enters the culvert. During 1927 in Barbour County 1,500 ft. of perforated pipe was laid in this manner. Slips that previous to placing the tile had required weekly attention have ceased to move. In Taylor County 500 ft. of perforated pipe was laid in this manner during 1927 with equally satisfactory results.

Usually during the winter and spring the clay banks slough off and fill the ditches with considerably more material than we can remove with the small forces that

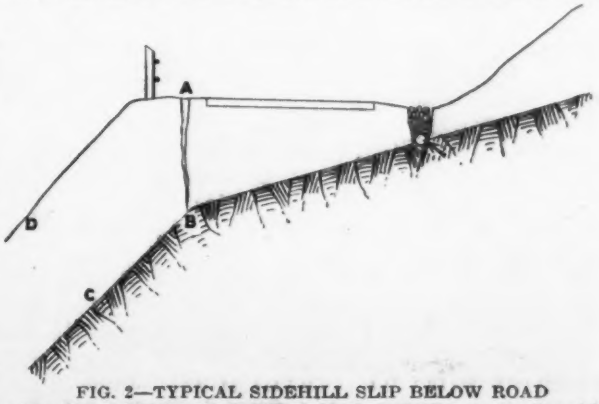


FIG. 2—TYPICAL SIDEHILL SLIP BELOW ROAD

The first indication is usually a crack occurring on the line *AB*. The material *ABCD* soon slips away and gradually slips from under the pavement. The beneficial results obtained by installation of drainage system as shown are usually apparent within 24 hours of installation. The material *ABCD* will cease to move within a few days, and backfilling to return shoulder to its original cross-section can be put in.

we have at this time of the year. Where these slips can be anticipated a perforated tile surrounded with broken stone is placed in the original ditch. The material falling into the ditch then does not block the waterway and cause water to flow over the road.

Boston & Maine Flood Damage Estimated Over $3,000,000

The New England floods of last November cost the Boston & Maine Railroad $3,314,000, according to a report to the stockholders by President George Hannauer. Of this, $2,314,000 is direct loss and $1,000,000 consequential loss. The report also states the new North Station in Boston will be completed by July 1.

Chute Handles 52,000 Cu.Yd. of Fill Across Railroad Tracks

Saves Contractors Long Haul in Moving Excavated Material From Cut to Fill on Pennsylvania Highway

DUMPING excavated material through a 120-ft. chute across a double-track railroad line is speeding up work and saving a 2½-mile haul on a road 9,700 ft. long, running west from the town of Kingston, Luzerne County, Pennsylvania. For about a mile west of town the road is on a low fill. Then, as shown by Fig. 3, it rises on a 5 per cent grade, passes over Toby's Creek and the Delaware, Lackawanna & Western Railroad tracks and swings around parallel to the tracks and the Susquehanna River on a slight gradient. The portion on the 5 per cent grade lying between the overhead bridge across the tracks and the Toby's Creek bridge, and a short section east of this bridge, required some 52,000 cu.yd. of fill, to be made from a 70,000-cu.yd. cut west of the overhead crossing.

A continuous haul from the cut to the fill meant a 2½-mile circuitous trip over an old road, crossing the railroad tracks at a busy colliery yard. To wait until the bridge over the railway was completed would have delayed the work at least six months. To overcome these objections the contractor built a chute 120 ft. long across the tracks, through which the excavated material is dumped from trucks on top to waiting wagons below. The material is loaded at the cut by two steam shovels into six 5-yd. dump trucks. Especially built 7-yd. wagons receive the material at the lower end of the chute, and crawler tractors haul the wagons the short distance to the fill. Two 10-ton tractors, hauling two wagons each, handle the material as fast as it comes from the shovels. The only other equipment used is a bulldozer at the fill handled by a 5-ton tractor. In wet weather a tractor can handle one wagon only, but the work is correspondingly slowed up at the cut, so no tie-up results.

The chute itself, built on a 33-deg. slope, is shown in Fig. 1. The 7x3½-ft. box rests on three 6x8-in. stringers running longitudinally. Twelve wooden bents of two 8x8-in. posts each and two steel frame bents carry the chute. Over the track the support is two 30-ft. I-beams resting on the two steel bents. This whole framework was salvaged from a wrecked building and suited the purpose without alteration. A clearance of 22 ft. is provided over the rails. The box is built of 2-in. lumber, lined with scrap sheet iron. At first a rectangular section was tried, but the material clogged at the corners, especially when wet, greatly retarding the flow. This condition was overcome by changing the inside of the box from a rectangular to a trapezoidal section.

No hopper is used at the lower end. The force of the descending material is spent against a backstop and the material falls gently into the waiting wagon. The backstop is a ¾-in. steel plate over a 4-in. timber mat, set normal to the flow line of the chute. The top of the backstop unit is fixed, while the lower side is buffered by four old railway coach springs set in hollowed out timbers. The whole is held and braced by one vertical and two inclined stub bents.

The truck runway at the upper end of the chute rests

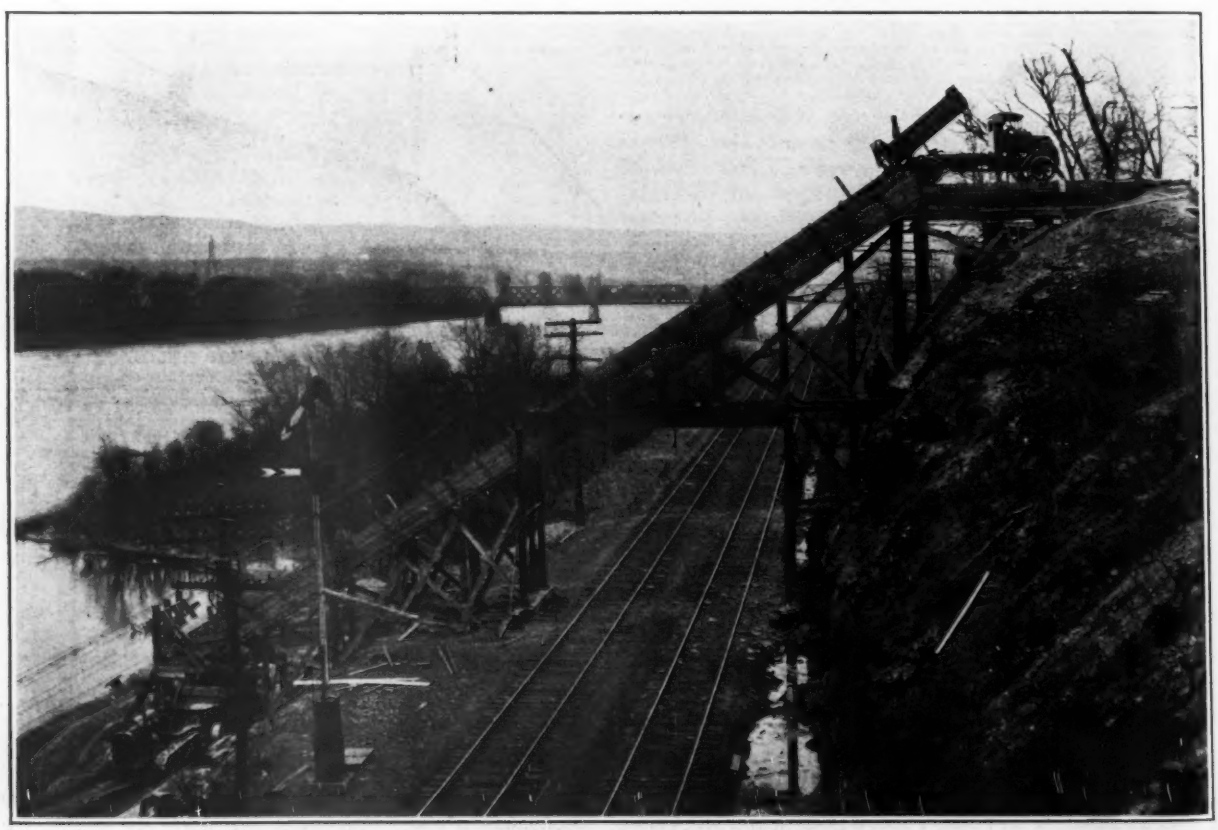


FIG. 1—CHUTE FOR TRANSFERRING EXCAVATED MATERIAL ACROSS RAILROAD TRACKS

FIG. 2—VIEW OF CHUTE SHOWING TRUCK DUMPING ON TOP INTO WAITING WAGON BELOW
Note steep rocky bank to right of railroad tracks, affording perfect foundation and anchorage for upper portion of chute.

on four four-post bents; the 4-in. plank floor is laid on 12x12-in. stringers. The floor has been raised twice by building up with 12x12-in. caps and stringers to secure an added height for the trucks when dumping. It was found that the material flowed better when given a short initial drop from the trucks.

The whole structure is very well braced and anchored. All bents are amply stiffened with sway-bracing. The steel frame is tied into the rocky bank by two 8-in. I-beams, supplemented by guy cables. More cables are used to hold the chute against the bank, and another set secures the truck runway.

Timing tests have shown only twelve seconds consumed from the time a truck starts to dump until the load is completely in the wagon below. A daily run of 1,000 cu.yd. has been put through the chute. A man at the top to guide the trucks as they back to unload is the only extra labor needed. There is no spillage below to clean up. The total labor and material cost of the chute was about $1,400. This amounts to approximately 2½ cents per cubic yard for the 52,000 cu.yd. required for the fill. Additional yardage from the 70,000-cu.yd. cut may be wasted through the chute, which will proportionately decrease this unit cost.

The chute was designed and built by H. C. Kersteen, president of the H. C. Kersteen Construction Company, of Kingston, Pa., which has the general contract for the project. P. W. Finn is superintendent on the job. C. H. Buckins is division engineer for the state highway department. All bridge and masonry work was sublet to the Anthracite Bridge Company, Scranton, Pa.

English Track Maintenance and Locomotives

Present relations between track and locomotive equipment on English railways were discussed in London recently at a joint meeting of the Permanent Way Institution and the Institution of Locomotive Engineers. From a brief report of the discussion in the *Railway Gazette*, London, the following summary is prepared, supplementing the editorial in *Engineering News-Record* of April 12, p. 577.

It was pointed out that the condition of rails is as important as their weight in judging their suitability for further service. Electric trains are hard on track, and the excessive wear is partly due to the abnormal grinding action of small driving wheels. However, it is principally multiple-unit trains which have small wheels; main-line electric locomotives usually have larger wheels, and one member referred to French electric locomotives having 69-in. driving wheels. Much of the trouble with electric traction was said to be due to the high acceleration and deceleration required in suburban service, while electric locomotives hauling main line trains do not have to meet these requirements on account of the different service.

Whether increase in weight of rails has kept pace with axle loading and traffic demand was considered open to question, while in view of excessive wear of rails under electric traction the use of hardened steel was advocated. Track maintenance should be considered for average service, rather than for rails and wheels in perfect condition. It was suggested also that in rail manufacture too much attention is being given to chemical and physical properties of the steel, and not enough to the question of obtaining better wearing properties. As to locomotives, although designers are developing efficiency as well as tractive power, it was thought that in both weight and power some modern engines are in advance of requirements. The question seems to be whether English locomotives are being developed with too great weight for the work they have to do or whether the track is being sufficiently developed to meet the demands of traffic and equipment.

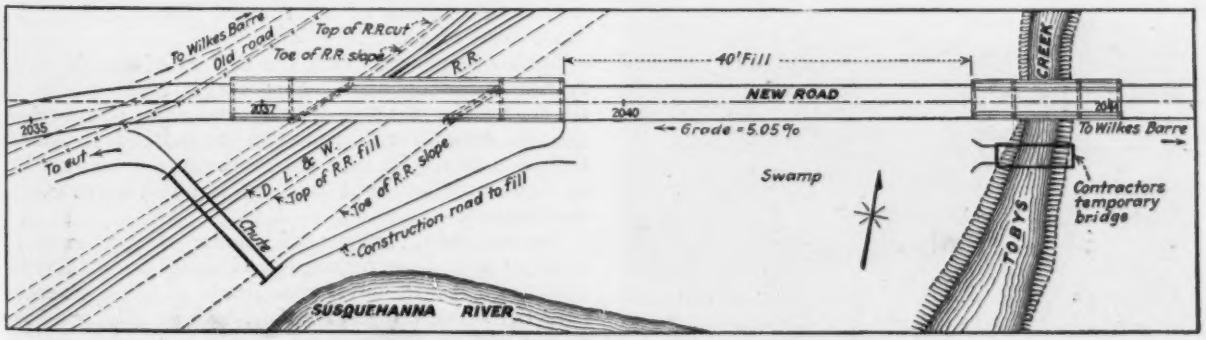


FIG. 3—LOCATION DIAGRAM SHOWING RELATION OF CHUTE TO THE WORK

Early Parabolic Truss Bridges Gradually Disappearing

History and Description of a Type of Truss, Some 300 of Which Were Erected in New England and New York Between 1880 and 1895

By Robins Fleming
American Bridge Company, New York

THE PARABOLIC truss bridge deserves a passing notice for its historic interest at least. Prof. G. F. Swain, one of the few who have given it this attention, mentioned in his recent book, "Structural Engineering—Stresses, Graphical Statics and Masonry," a form of truss with both chords curved and supported on verticals at the ends, "largely used a few decades ago for highway bridges by the Berlin Iron Bridge Company." The writer was for a considerable period of time connected with the Berlin company and designed scores of these bridges. By this article it is hoped to place on record something of the history of these interesting structures, which once were quite common in New England and New York but now are rapidly disappearing, and in addition to note a few of the design considerations.

History—A patent dated April 16, 1878, was granted to William O. Douglas, Binghamton, N. Y., for a truss bridge, described in the patent application as "A combination of two or more elliptical trusses connected as herein described with the floor and joists and the necessary flooring to form a through, deck or swing bridge." Two or three of these bridges had been built by the Corrugated Metal Company, a small manufacturing firm at East Berlin, Conn., but they were crude affairs, and not until the company acquired the services of Charles M. Jarvis, of Binghamton, as chief engineer, did the company flourish. The name of the company was changed to the Berlin Iron Bridge Company, and on the death of the president in 1886, Mr. Jarvis became president, retaining that position until 1900, when the company, along with 25 other bridge companies, was merged into the present American Bridge Company.

FIG. 3—DECK PARABOLIC TRUSS AT SUFFIELD, CONN.

Soon after becoming chief engineer Mr. Jarvis changed the shape of the chords so that the pins were placed at points of true parabolas and the name parabolic trusses soon became accepted.

Between the years 1880 and 1895 some 300 or more of these bridges were built, mostly in New England and New York. Many of them still remain and are carrying present-day traffic, although in most cases such bridges have probably been reinforced. The following are some of the larger parabolic bridges of which there is still a record: Lowell, Mass., bridge over the Merrimac River, of five 153-ft. spans with a 32-ft. roadway and two 7-ft. walks; Nashua, N. H., bridge over the Merrimac, of three spans of 150 ft., 160 ft. and 170 ft. respectively; Manchester, N. H., bridge over the Merrimac, of three spans of 140 ft., one of 55 ft. and two of 64 ft.; Shelburne Falls, Mass., a single span of 198 ft.; Waterbury, Conn., a single span of 205 ft.; and Binghamton, N. Y., three spans of 165 ft.

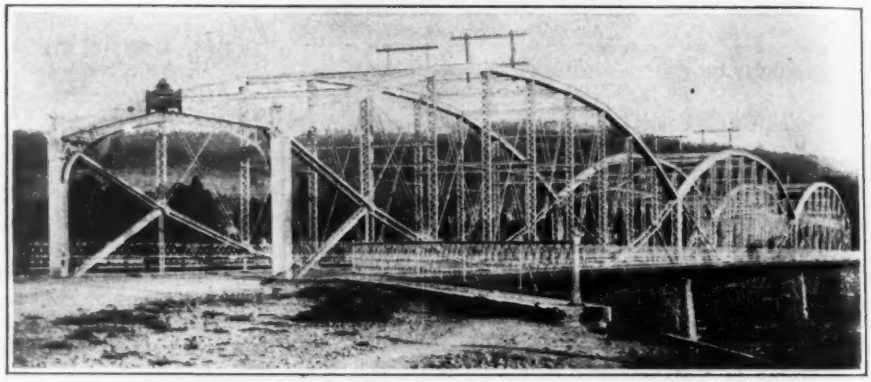


FIG. 1—THROUGH PARABOLIC TRUSS BRIDGE AT BINGHAMTON, N. Y., WITH THREE 65-FT. SPANS

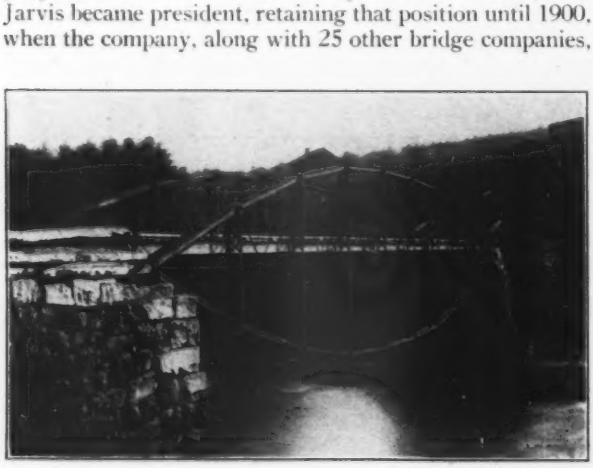


FIG. 2—PARABOLIC TRUSS OF SEMI-DECK DESIGN

With a single exception, the bridges of the parabolic type were for highways. Few were built after 1895, as the demand for this type of bridge had about disappeared. Another reason for this was that the Berlin Iron Bridge Company during its later years stressed the construction of steel-frame industrial buildings, in which it was one of the pioneers.

Several noted bridges of similar design exist in Europe; there, however, the truss is called the Pauli truss from its inventor. Professor Swain in his book mentions the Rhine bridge at Mainz, built in 1857 and consisting of four 345-ft. spans, as the principal bridge of this type. Another similar bridge is the Royal Albert

bridge, which carries the Cornwall Railway across the River Tamar at Saltash, England. This bridge was designed by I. K. Brunel and was opened in 1859. It has a total length of 2,200 ft. and contains two 455-ft. spans over the river and seventeen side spans varying from 70 to 90 ft.

Design—The parabolic bridge presents a pleasing appearance, especially when of three or more spans. It was easily designed, since each chord was of the same cross-section throughout—the top chord being of two channels with plates on top and bottom chord of eyebars. Trusses were invariably pin-connected. The stresses in the web system were small, allowing the diagonals to be round bars with loop eyes and turnbuckle or sleeve-nut adjustment. The posts were four latticed angles, and the floorbeams were hung from pins by inverted U-suspenders of square bar iron, whose lengths could easily be varied to give any desired camber. Few working drawings were needed, and fabrication was simple.

An objection to the parabolic bridge was its lack of stiffness. The entire floor system was suspended from the lower chords by bar hangers, some of which were often several feet long. As the height of the end posts was about one-half the center depth of truss, an upper lateral system of bracing could be carried only to the tops of the end posts. It was therefore often necessary, in spans of about 100 ft., to end the upper lateral system at a portal moved far enough back from the end posts to allow sufficient headroom. To carry the lateral force to the abutments, dependence was placed on the rigidity of the chords and end posts.

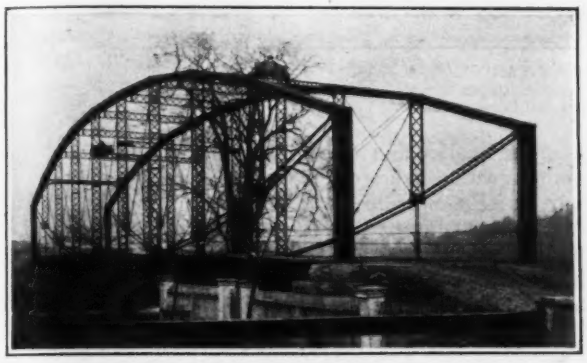


FIG. 4—PARABOLIC TRUSS WITH PORTAL AT FIRST PANEL POINT

It was often necessary to set the portal back from the end posts in order to provide adequate headroom between the bridge floor and the upper lateral system.

Highest Tides in the World Found Along the Atlantic Coast

The highest and most variable tides in the world are found along the Atlantic coast. According to Atlantic Coast Tide Tables for 1929, latest compilation of tides for that region made by the Coast and Geodetic Survey, they range from 50 ft. in the Bay of Fundy in Canada to only 1 ft. at Nantucket Island off Rhode Island. Utmost variation in tides is found along the east coast. New York harbor, for instance, has a tidal range of about 4½ ft., whereas Eastport, Me., a few hundred miles away, has a range of 20 ft. Boston's tide rises 10 ft., while at Key West, Fla., the ocean has a tidal range of 1½ ft.

New Code Lessens Traffic Hazards in San Francisco

Lower Accident Toll Results From Pedestrian Control, Traffic Fines Bureau and Other Features of Ordinance

AFTER several months' experience with a new traffic code in San Francisco, figures showing a notable reduction in the number of personal injuries and in the amount of property damage are announced by the traffic survey committee. "A saving of $2,000,000 a year in the cost of automobile accidents is being made for vehicle owners of San Francisco by reason of the new traffic regulations," the committee said. Reductions in accident frequency since the code went into effect range from 30 to 40 per cent in the records of three companies operating 50, 100 and 400 motor vehicles, respectively. The Market Street Railway Company reports a reduction of 24.7 per cent in pedestrian accidents and coroner's records show no fatal accidents in the central traffic district since the ordinance went into effect.

The street traffic control problem of San Francisco was the subject of a report compiled last August by the San Francisco traffic survey committee, which is an organization supported by contributions from civic bodies. This report was based on an engineering survey by Miller McClintock, a traffic expert, retained to make a study of traffic flow, volume, speed, concentration and accidents, together with an analysis of the data collected with respect to accident hazards.

As the result of this report, a new traffic ordinance was formulated and there was created a traffic fines bureau and a division of street traffic engineering for which George D. Burr was appointed city traffic engineer. A traffic law enforcement board was organized in which are representatives of the California State Automobile Association, the San Francisco traffic survey committee, the managing editors of five daily newspapers, four police court judges, the board of supervisors, coroner, district attorney, chief of police and the city traffic engineer.

The new traffic code went into effect Nov. 1 last and the traffic fines bureau began to impose penalties about Dec. 1. Included in the new ordinance are provisions for: (1) The segregation of pedestrian and vehicular traffic; (2) educational campaigns in the newspapers and other mediums to acquaint the public with the new regulations and the mutual responsibilities of the various elements that constitute the traffic; (3) the traffic fines bureau, which assures swift and just punishment for traffic violations, and (4) the study and remedy of the conditions at particularly hazardous locations where accidents are been frequent.

Although speed limits have not been raised, the new ordinance has materially increased the average traffic speed by eliminating unnecessary delays at clearance zones near intersections and by providing loading zones to eliminate double parking. The control of stop and go signals has been co-ordinated so that where possible traffic is allowed to move uninterruptedly through several intersections.

Further proposed improvements in the traffic control system include a separate signal system for the control of pedestrian traffic on Market St. and a new scheme for reporting and analyzing traffic accidents.

High Earth Dam Threatened by Washout

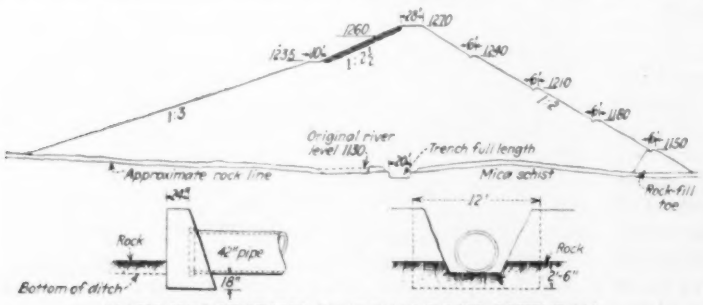
New Water-Supply Dam of Greenville, S. C., 140 Ft. High, Endangered by Failure of Outlet Pipe—Slow Drainage of Pool Will Leave Structure in Critical Condition for Some Time

EARLY VIEW OF WASHOUT AT TABLE ROCK DAM
Looking north from abutment. Slide at Pickens County end, with washout gorge just beyond its north edge. Displacement can be gaged by lines of berms.

ON MAY 4, a new storage dam of the Greenville (S. C.) water-works developed a sudden washout of the downstream toe in the highest section, and soon was in a condition that threatened immediate failure. The washout occurred just over a 42-in. cast-iron drainage pipe passing through the dam at stream level; in a short time it cut a deep gorge in the downstream side of the embankment, extending back about 100 ft. from the toe and bringing down large masses of earth. Simultaneously a section of the downstream slope to one side of the washout came down in a great slide, bringing up against a projecting spur of the valley-side slope.

The reservoir was nearly full at the time of the washout and is now draining slowly. As it is of exceptionally large capacity (9 billion gallons or 28,000 acre-ft.), the embankment will remain in a critical condition for at least a week or two. By Monday evening, May 7, the weakened embankment had held with no substantial change for nearly three days in spite of two days of heavy rainfall (about 2 in. fell in less than 30 hours), and the prospects appeared excellent for saving the situation.

Sketches of the dam in cross-section and plan made from construction drawings shown herewith. On the plan the location and shape of the washout and slump are approximately indicated, from inspection by a representative of *Engineering News-Record* on May 6 and 7.



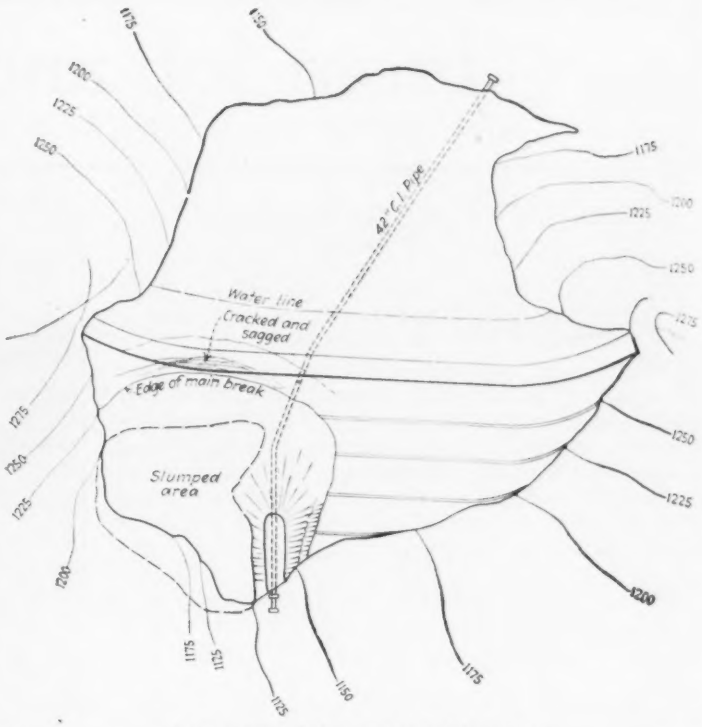
MAXIMUM SECTION OF TABLE ROCK COVE DAM
Detail shows downstream end wall of drain pipe and general form of trench in which pipe was laid. Cutoff walls of concrete 18 in. thick were spaced at intervals of 50 ft. or more along the pipe.

The dam is in the valley of the South Saluda River on the southern slope of the Blue Ridge Mountains, about 32 miles northwest of Greenville, at Table Rock Mountain, and is referred to as the Table Rock Cove dam. It was built between 1925 and 1927 to supplement the present Paris Mountain water supply of Greenville and environs. J. L. Ludlow, of Winston-Salem, N. C., was engineer of the new supply and designed the dam. Rinehart & Dennis, of Charlottesville, Va., built the structure. The dam contains about 600,000 cu.yd. of earth, forms a reservoir of about 800 acres maximum area and has a crest height of 140 ft. above original river level. It is a rolled-fill embankment, and was built by dumping earth from trains of cars, spreading it with road graders in 6-in. layers, and compacting thoroughly by rolling. The material contains no gravel, but is a sand-clay mixture of residual soil derived from the granite or gneiss country rock. Longitudinally of the embankment there is a shallow cutoff trench excavated in hardpan and rock for secure bond between structure and foundation. The upper part of the water slope is paved with riprap.

The 42-in. drain pipe was laid in a trench cut in the earth and rock valley floor close to the streambed and served to divert the stream flow during construction of the embankment. It was not intended for normal use subsequently, but was closed off by a 36-in. gate valve just above the downstream end, placed within a valve vault. Transverse cutoff walls of concrete, 18 in. thick, notched into the bottom and sides of the pipe trench, were spaced 50 ft. or more along the pipe.

The reservoir has never been filled completely, but in recent weeks the water has been rising and last week reached a level

about 15 ft. below spillway crest. No leakage through the embankment or other seepage from the reservoir had been noticed at any time, but during recent weeks there was some seepage from springs at the toe and the south abutment. One of these, more than half way up the south abutment, had clogged a drainage ditch provided along the embankment in the natural hillside, and before the ditch was drained a small patch of the end of the dam had softened; upon drainage of the ditch overflow the softened spot soon dried out again. Other slight spring flows came from an old spring of small volume under the downstream side of the embankment near the south or Pickens County abutment, which during construction was provided with a drain to the toe; from the foundation of the valve vault, which was located on a wet spot or spring; and from a point in the natural ground of the north or Greenville County abutment. Because of the clogging of the south abutment ditch (noticed on April 22 and corrected on April 23), these points were the subject of careful examination by H. W. Perry, superintendent of water, and J. L. Ludlow, engineer of the dam, on April 23 and April 25; in their opinion these seepages have no connection with the subsequent developments, and they state that no other happenings were observed until May 4.



PLAN OF TABLE ROCK COVE DAM

From sketches made from construction drawings. North or Greenville County end at right, Pickens County end at left. Spillway in natural saddle just to right of sketch. Outline of slumped and washed-out areas roughed in from field sketches.

Late Friday afternoon, May 4, the condition of the dam was observed to be entirely normal, but within half an hour after this observation the keeper heard a noise and saw a geyser-like burst of water and earth coming through the downstream toe a little above the valve vault. He tried to open the gate valve, but did not succeed, and then sent a messenger 5 miles to the nearest telephone. Mr. Perry, on reaching the dam, found a large flow of water gushing out of an opening around the lower end of the outlet drain. By the next morning this cave had developed into a gorge-like trench and the downstream face had slumped almost as in the sketch. In the following two days the situation developed little change except sliding down of some minor loosened masses and possibly a slight increase of settlement in the crown of the dam just back of the slump, where there is a series of concentric cracks, the earth downstream of which had sunk in blocks or slices by amounts ranging from a few inches to a foot or more. This part includes a width of 10 or 20 ft. outside the upper edge of the slide, which latter almost reaches the crest of the downstream slope. Traces of other concentric cracks lying as much as 10 or 15 ft. outside the outer limit of the principal cracked zone can also be seen in the crown of the dam, and one of these was definitely visible in the upstream slope at a level 6 or 8 ft. below the crown.

Associated Press Photo

WASHOUT GORGE FROM BELOW

It was believed from the first that the source of the washout was a break or opening in the drain pipe some 60 or 70 ft. inward from its lower end. As the washout gorge developed, a marked outflow wave and turbulence appeared on the surface of the outflowing water at about the location described. The appearance indicated quite definitely that the flow from the pipe was concentrated in one spot, as might result if a joint had pulled apart or a break of some size had developed in the pipe.

Drainage of the reservoir is proceeding by way of the drain pipe and the 30-in. service outlet located near the Greenville County side. The water department broke a hole in the service pipe line a quarter mile below the dam to provide a vent, wasting the water back into the river channel. The combined outflow was lowering the water by about 2 ft. per day.

Latest Condition—Supplementing the preceding, based on the condition prevailing Monday, May 7, the following report of condition on Tuesday, May 8, was received from Colonel Ludlow at the time of going to press:

Conditions at dam this (Tuesday) evening about same as yesterday. Slightly more settlement on crown at south end. Signs of cracking and settlement in crown extending farther toward center. Lake level continues lowering though somewhat retarded by rainfall. J. L. LUDLOW.

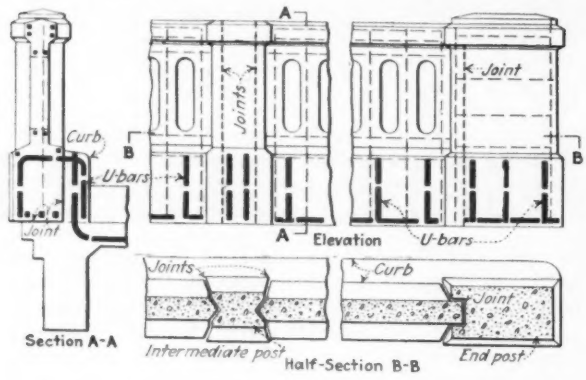
From Job and Office

Hints That Cut Cost and Time for the Contractor and Engineer

Movable Railings for Reinforced-Concrete Bridges

By Alfred R. Mellor
Assistant Structural Engineer, Massachusetts Department of Public Works, Boston

SINCE it is impractical to build all roads wide enough for possible future development, it is equally impractical to build all bridges as wide as they may ultimately be required. What is needed is to construct every bridge in such a manner that it may be easily widened or changed to meet future requirements without destroying any part of the original structure. Such considerations make a design for a removable bridge railing desirable. The accompanying sketches show a design for a bridge railing which may be removed and replaced on the widened bridge without injury and which in case no widening becomes necessary will remain a permanent part of the original structure. It is believed to provide as rigid a railing as other designs now used for bridge railings and to be no more expensive and no more difficult to construct. In view of the fact that the steadily increasing volume of traffic will necessitate many changes in bridge widths, such a fence should result in a worthwhile saving.



DESIGN OF REMOVABLE CONCRETE BRIDGE RAILING
To detach railing, remove curb.

Concrete Skeleton Tetrahedrons Prevent Bank Erosion During Floods

THE use of concrete skeleton tetrahedrons has effectively prevented flood-water bank erosion in the Santa Clara River near Montalvo, Calif., according to *California Highways and Public Works*, the official bulletin published by the California Department of Public Works.

The abutments of a 2,077-ft. highway bridge on the Los Angeles-Ventura highway were threatened by every flood before the tetrahedrons were placed. A large amount of driftwood and brush is carried by the Santa Clara River during flood times. The row of concrete tetrahedrons laced together now catches a lot of this

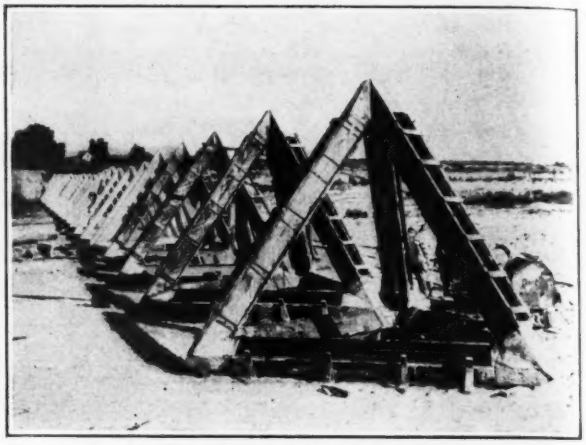


FIG. 1—ROW OF TETRAHEDRONS IN PLACE

débris. As the brush accumulates, reducing the velocity of the water at that point, silt and sand are deposited. In this manner the river builds its own barrier and later trees and brush grow up in the deposited silt, making the barrier more effective. Even if a unit should be toppled over by the flood, it is still resting on a base that is equal in area to its original one, resisting the flood as before.

Construction of the tetrahedrons was carried on dur-

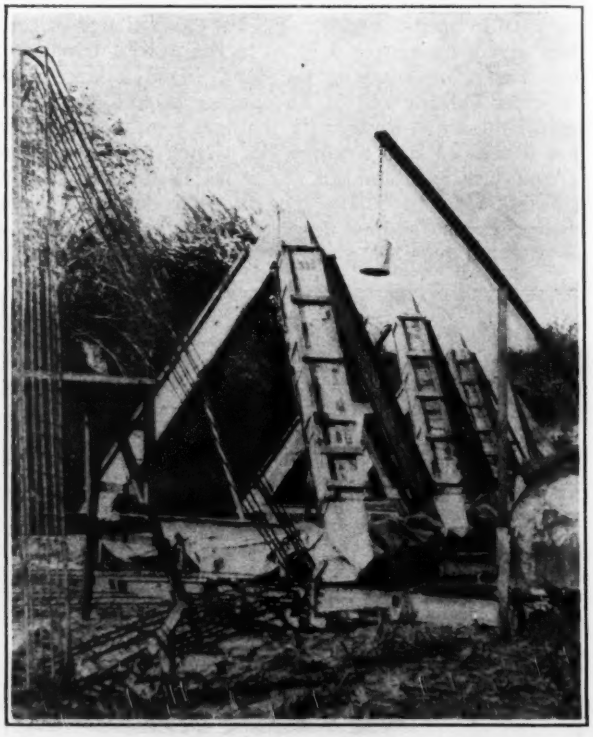


FIG. 2—FORMS READY FOR CONCRETE
Note use of teeter poles to place concrete.

ing low-water period. Figs. 1 and 2 show the details of construction. The tetrahedrons were made up of six legs, 12 in. square, 16 ft. long, reinforced with eight ½-in. bars. The two outside corner bars of each leg were 17 ft. 6 in. long, bent and tied into the opposite corner bar of the adjacent leg, with six straight bars 14 ft. long used in each leg. A No. 8-wire spiral reinforcement, 6 in. pitch, was wound around the bars. The wood forms were supplemented by sheet metal at the corners.

The individual units of the row of tetrahedrons were lashed together as shown in Fig. 3, spaced about 3 ft. between corners with the 16-ft. face lined up toward the upstream of the river. Old 1-in. cable, secured from a near-by oil field, was used to tie the units together. The shore ends of these cables were anchored in concrete blocks.

The tetrahedrons were constructed at a cost of about $112 each. The cost of the protection work, including

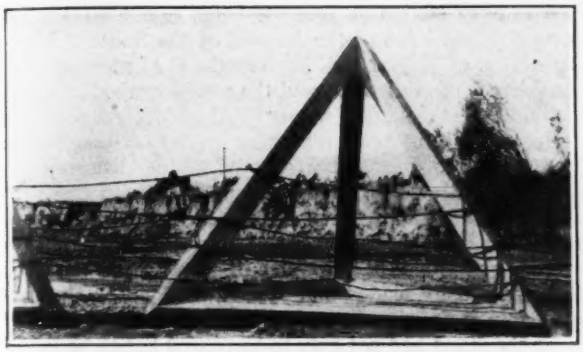


FIG. 3—A COMPLETED TETRAHEDRON
Six 1-in. cables clamped to legs hold units together and to shore anchor.

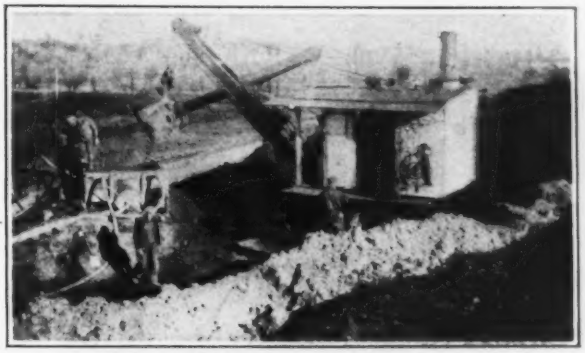
all expenditures, was a little more than $7 per foot. The work was performed by day labor in charge of Roy Stover, foreman.

The description of these tetrahedrons was furnished to the bulletin by E. T. Scott, assistant district maintenance engineer, California Department of Public Works.

Inexpensive Hydraulic Fill of Swamp Makes Valuable Parking Space

THE County of Ramsey has acquired a bathing beach at McCarron's Lake just north of St. Paul, Minn. The number of people using this beach last summer passed all estimates, and it became necessary to increase the open space and parking area adjoining the beach. This land is low and swampy, so filling was necessary, though funds for the work were limited. A suitable borrowpit was secured immediately south of this swamp land, the county road intervening. A power shovel with a capacity of 1,500 cu.yd. a day was placed in the pit. Unusual fall rains made it impracticable to use trucks, and in order to complete the fill at once instead of waiting for the water to dry out, the county engineer, Jaul N. Coates, adopted the following method:

Two 6-in. wrought-iron water mains were run from a city hydrant to the borrowpit and a wooden hopper was built for the shovel to dump dirt into. Two 3-in. fire hose were taken off each 6-in. water main and four 1¼-in. nozzles inserted in the end of the hopper. A 12-in. pipe



SHOVEL DEPOSITING INTO HYDRAULIC HOPPER

line supported on stilts was run from the hopper to the swamp, clearing the county road about 12 ft. and having a gradient of 3½ ft. to the hundred. The length of the pipe line ranged between 600 and 1,000 ft. The shovel deposited 1,500 cu.yd. per day into the hopper, which had a grating over the top made of 2½-in. angle iron spaced about 8 in. apart to keep out occasional boulders. The four nozzles delivered 1.5 m.g. of water to the hopper, breaking up the dirt into a fluid state and transporting it to the swamp. The water carries approximately 20 per cent dirt.

The cost of placing the plant in operation, including all costs of pipe, was about $700, and the daily cost of operation is $62, which includes cost of water at $18 per million gallons. About 60,000 yd. was required to make the fill. The cost of moving this dirt averaged less than 5c. per cubic yard.

Extra-Long Profile Sheets Handled on Reversible Roller Cabinet

By W. E. Elam
Assistant Engineer, Board of Mississippi Levee Commissioners, Greenville, Miss.

A SPECIAL roller cabinet was devised by the author to handle the long profile sheets used by the Mississippi Levee Board engineers at Greenville, Miss. It is necessary to correct and work on the profiles year after year to keep a permanent accessible record of high-water elevations. Ordinary methods of handling would soon ruin the sheets. The case, 5 ft. 6 in. long by 2 ft. 5 in. wide, is mounted between two standards so as to rotate in a vertical plane. A removable glass top protects the profile while not being worked on. (Fig. 2.)

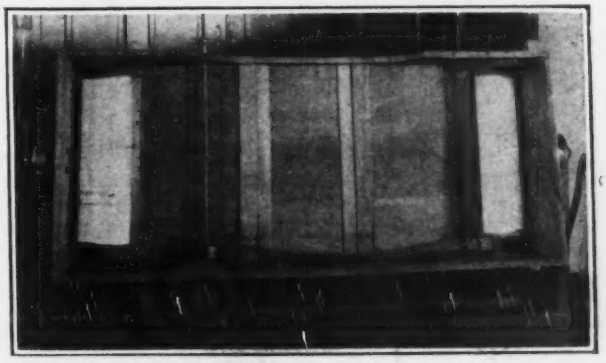


FIG. 1—BOTTOM VIEW OF CABINET SHOWING ROLL DRIVING MECHANISM

FIG. 2—PROFILE CABINET IN USE
Glass cover raised to permit working on sheet

The paper moves through guides over a smooth drawing board to the storage rollers underneath. Idler rollers at each end of the board keep the paper from tearing. The storage rollers are wooden spools just wide enough to receive the 22-in. paper and act as power rolls to wind and unwind the sheet.

To permit one roll to run idle while the other was winding up, the clutch detailed in Fig. 3 was devised. From Fig. 3 it can be seen the power is transmitted from a hand crank at the front side of the case through the clutch to the rolls by an ordinary bicycle chain. The clutch consists of one driving disk keyed to the shaft, engaging one of two driven disks through two dogs. Only one set of dogs and one driven disk can operate at one time. Reversing the crank releases one set of dogs, engaging those on the opposite side. The driven disks are fastened direct to the sprocket carrying the chain. A light brake around all three disks insures the clutch releasing when reversed and also keeps the paper taut on the rolls.

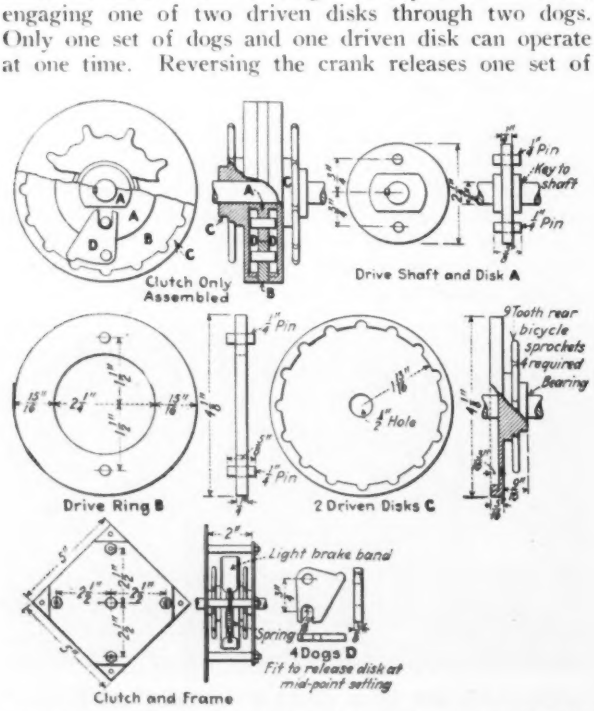


FIG. 3—DETAIL OF DRIVING CLUTCH FOR PROFILE ROLLS

The capacity of the case is limited only by the size of the storage spools. The present profile in use is 40 yd. long.

From Job and Office

Hints That Cut Cost and Time

Reinforced-Concrete Grandstand Moved on Rollers

A REINFORCED-CONCRETE grandstand weighing 290 tons and 75x47 ft. in plan was moved horizontally a distance of 241 ft. recently to make room for the new railway station at Johannesburg, South Africa, reports a recent issue of the *Journal* of the South African Institution of Engineers. Two methods of moving were considered, the rail track and the wheel method, and the method herein described was decided on.

Rollers of 4-in. steam pipe were used, first being tested in a hydraulic press. No sign of undue distortion or flattening with a load of 4 tons appeared with this size. Twenty-five rollers were placed at first under each transverse row of columns at about 2-ft. centers (175 rollers in all), giving a load per roller of about 3,300 lb. when all were equally loaded and spaced, a condition that existed only at the start.

As the stand had not only to be moved but to be raised a distance of 10 ft. 6 in., the tracks were on an inclined plane—that is, the stand was raised as it was moved forward.

The first step was the excavating of material underneath the old stand to about 12 in. above the level of the bottom of the timber tracks, the ground being left intact under the columns. Each transverse frame section was then supported on timber struts, one at a time, splayed out at the bottom in the form of the two legs of the letter "A," the ends being placed on sleepers and wedged to take the weight. The bottom of the south column was then cut to conform to the angle of the straight timber cradle. A trench about 4 ft. wide by 50 ft. long was then dug under the three columns, the bottom of the trench being about the level of the under side of the roller track.

Short sleepers were sunk in the bottom of the trench at from 12-ft. to 15-ft. centers and accurately leveled for the grade, the ground flattened out between the sleepers and well tamped level with the top. The track, consisting of two 12x4-in. timbers laid on the side, was then laid in the trench and the 25 rollers were placed on the track at about 2-ft. centers. The 12x12-in. cradle timbers were then threaded through between the rollers and the foot of the concrete columns. The spaces between the top of the cradle and the bottom of the concrete columns were next filled in with timber, the top of this timber being carefully shaped to suit the irregularities of the concrete footing. Double hardwood wedges driven against each other were then driven in between the cradles and the timbers until the weight of that section of the stand was taken on the cradle, rollers and track. The splayed timber struts were then removed.

This procedure was carried out in each of the seven

From Job and Office

For Contractor and Engineer

transverse frames, so that when completed the whole concrete structure was carried on rollers. Care was taken to insure that all the roller tracks were in the same plane.

Substantial yoke bolts were fitted at each end of each main truss to tie the trusses to the north and center columns, and to tie the half truss into the south columns. This was done because these trusses were precast and any undue distortion of the structure might have resulted in the ends of the trusses becoming loosened from the columns. The lower ends of the columns were tied together longitudinally by double 12x3-in. timbers, one on each side of the concrete columns, with through bolts close against the columns on each side. Each transverse frame was then strutted and braced with 6x6-in. timbers.

The actual moving was done by two 10-ton triple-geared hand winches bolted to a heavy framework of timber anchored in the ground and bags of sand used as ballast. Two sets of two and three pulley blocks were used, one for each winch, giving a six-rope pull on each set of tackle.

Recording Concrete Strength Aids in Getting Better Bridges

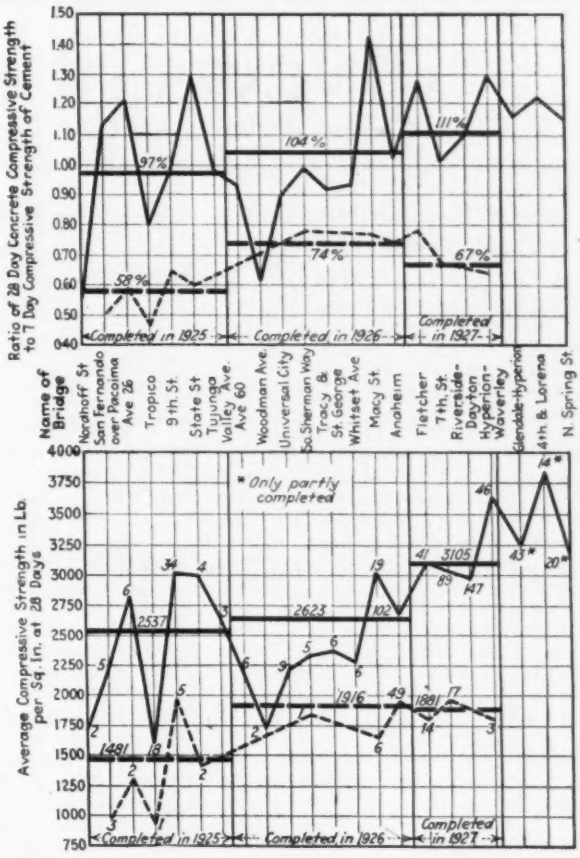
FOR SEVERAL years the city of Los Angeles has made a practice of keeping accurate record of the concrete strength average on the various concrete bridges built for the city by contractors. Constant testing of concrete samples that are taken without any forewarning has resulted in the development of improved methods of proportioning, mixing and handling the concrete. A steady and material increase in the average strength of concrete has resulted, extending over the period of more than three years in which the plan has been in effect.

The accompanying diagrams show graphically how the strength on the several jobs has increased from year to year. During 1925 the average of tests on Class A concrete (weighted to allow for the varying number of samples from each job) showed an average strength of 2,538 lb. per sq.in. in 28 days. In 1926 the corresponding average was 2,263 lb. per sq.in., and in 1927 it was 3,105 lb. per sq.in. The ratio of compressive strength in 28-day concrete to compressive strength in seven-day cement also shows a corresponding increase year by year, the ratio for 1925 being 97 per cent, 104 per cent in 1926, and 111 per cent in 1927.

In both diagrams the upper curve, plotted as a light solid line, denotes Class A concrete (1 : 2 : 3½) and the lower curve, plotted as a dotted line, is for Class C concrete (1 : 3 : 5). The small figures at each point on the curve indicate the number of tests on each bridge. Averages for each year are shown by the heavy horizontal lines. In no cases were cylinders that gave low-strength breaks eliminated from the averages.

Blueprints containing the information shown on the accompanying diagrams are made up for the Los Angeles engineering department in 3x5-in. sizes so they will fit into the ordinary card indexes. This has been found to be a convenient form in which to supply these data to contractors and engineers who have city bridge work under way or are interested in bidding on new jobs.

For the foregoing information *Engineering News-Record* is indebted to R. W. Stewart, chief deputy city engineer under John C. Shaw, city engineer of Los Angeles.



DIAGRAMS SHOWING RESULTS OF CONCRETE STRENGTH STUDIES ON LOS ANGELES BRIDGES

Job and Office Notes

HYDRAULIC REMOVAL OF LANDSLIDES from California highways has been practiced extensively in Humboldt County by District Engineer T. A. Bedford. The particular slide which has caused the most trouble is known as the Big Blue Slide and was of such area that in places there was not even an appearance of a highway left. The slide area is a third of a mile long, a fourth of a mile wide, and 400 ft. high. The material is disintegrating blue shale, marked with cracks and fissures. It is estimated that there will be 200,000 cu.yd. of this material to move in the next few years. In order to move it, a 4-in. pipe line was brought from Fish Creek with a 200-ft. fall and a distance of half a mile from the slide. Two deep trenches are sluiced into the mountain side, and so arranged that they converge into a sluiceway. These trenches catch the slide as it comes down the mountain side and carry the material into the sluiceway, which is a 6x5-ft. box with a lip on the upper end. These trenches drain the water out of the slide. This particular sluiceway was on a 1:4 slope, but Mr. Bedford recommends a 1:3 slope so that the slide will automatically dispose of itself without sluicing. The head on the pipe line is sufficient to move the material into the sluiceways hydraulically without pumping. Three men moved 300 cu.yd. of material per day at a total cost of 10c. per cu.yd. Mr. Bedford reports that when the trenches are dug and the slides trained to follow them, little care is necessary. Last winter 25,000 cu.yd. of material went through one of these sluiceways with very little attention.

News of the Week

Current Events in the Civil Engineering and Contracting Fields

Report Made on New York Rapid Transit Problem

The Regional Plan of New York and Its Environs has issued the first section of its report on "Transit and Transportation," which presents the findings and conclusions of a large corps of experts in economic research, engineering and community planning. The conclusion reached is that the expenditure of a billion dollars for subways in the next fifteen years will not relieve congestion unless some steps are taken to decentralize the industry and commerce now concentrated in lower New York.

The report states that subways have never been able to keep pace with the demand for their use and that the building of new subways invariably creates new centers of congestion, increases the riding habits of the public and makes for more crowding of all transit facilities. "Instead of trying to bring more workers to Manhattan daily," says the Regional Plan, "we should bring the place to work closer to the home of the worker."

On a typical business day nearly 3,000,000 persons enter that part of Manhattan south of 59th St., and about 2,000,000 of these use the subways and elevated trains. The practical standstill in subway construction from 1920 to 1926 has resulted in congestion which is "indecent, distressing and degrading."

Other facts brought out in the report are: (1) That there are, in New York City, more than 200 transportation companies holding over 1,000 franchises, many of these for long terms, and a number of them perpetual. (2) That in the region served by subway and elevated lines there were, last year, about 548 fares paid per capita. (3) That the average haul on these lines has been from 5.5 to 5.8 miles, remaining practically constant since 1911.

Cotter, Ark., Dam Permit Granted by Power Commission

A preliminary permit for the White River Power Company, Little Rock, Ark., was authorized at a recent meeting of the Federal Power Commission. The company's plan provides for the construction near Cotter, Ark., of a storage dam 226 ft. high. Other dams and power houses below Cotter on the White River and on its Buffalo and North forks are planned. It is estimated that 70,000 hp. of primary power will be made available. It is planned to install 165,000 hp. A conflicting application by the Ozarks Hydro-Electric Company was withdrawn recently. (See *Engineering News-Record*, March 22, 1928, p. 492.)

Cascade Tunnel Holed Through

By the touch of an electric switch in Washington, President Coolidge, on May 1, set off the blast which holed through the pioneer bore of the Great Northern Railway's 7.79-mile tunnel through the Cascade Mountains 100 miles east of Seattle, Wash. The tunnel, which will cost $16,000,000, will be the longest in the Western Hemisphere and the fifth largest in the world. The first train is expected to go through in December.

Another Million Dollars for St. Francis Rehabilitation

The Los Angeles City Council on May 3 transferred $1,000,000 from funds of the Municipal Power Bureau for use in rehabilitation work in Ventura County in the area damaged by the flood following the break of the St. Francis dam. The first $1,000,000, appropriated soon after the break, will probably be expended by the middle of May, and this additional appropriation should suffice to continue the work for another month, by which time it is hoped that some plan for financing the damage cost in its entirety, estimated at about $10,000,000, will have been adopted. Allocation of funds by the city council from available sources or the passing of a bond issue are the alternatives under consideration.

Cave-In Buries Equipment in Asheville Tunnel

Timbering near the western portal of the Beaucatcher road tunnel under construction in Asheville, N. C., proved unable to hold the moving earth, and it collapsed on April 24. The earth had been gradually slipping for some weeks and several times threatened to stop the work. Warning of the cracking timber gave the workmen time to escape toward the eastern portal. The power shovel, several flat cars and other tools were buried in the débris. Attempts to remove the sliding earth over the tunnel had proved unsuccessful.

The tunnel is of unusually large section and the greater part of it is through rock. The excavation section is approximately 48x27 ft. The tunnel is to be used to give a direct entrance to the city for road traffic from the east. As described in *Engineering News-Record* March 15, 1928, p. 441, unusual care had been taken to prevent slides at the west portal. The contractor is the Clement-Dunevant Company, Knoxville, Tenn., and the Howerton Engineering Company, Asheville, is the engineer.

American Engineers to Attend Two Functions Abroad

The Institution of Civil Engineers, of England, will celebrate the 100th anniversary of the granting of its charter during the week beginning June 3. The American Society of Civil Engineers has designated Allen Hazen, past vice-president, and George T. Seabury, secretary, as its official representatives. Other American technical societies will also send official delegations.

On July 4 the Louvain Memorial to American engineers who lost their lives in the World War will be dedicated. A large number of engineers will attend, both as members and as representatives of the technical societies. Members of the committee who will be present are: Edward Dean Adams, chairman, representing the American Institute of Electrical Engineers; Arthur S. Dwight, representing the American Institute of Mining and Metallurgical Engineers; Arthur W. Berresford, representing the American Engineering Council; and Capt. L. R. Lohr, representing the Society of American Military Engineers. The attendance from the American Society of Civil Engineers will include, as official delegates, George S. Davison, past-president; George T. Seabury, secretary; William J. Barney, and William K. Hatt. Representatives will also be sent by the American Institute of Electrical Engineers, American Society of Mechanical Engineers, American Institute of Consulting Engineers and the Society of Naval Architects and Marine Engineers.

Largest Building in the World Planned for Chicago

Plans have been completed for the construction in Chicago of a building more than louble the size of any present structure. This building, to be known as the Merchandise Mart, will be erected on a rhomboid-shaped plot on the Chicago River opposite Wacker Drive. It will have a frontage along Kenzie St. of 724 ft., along Wells St. of 324 ft. and along the river shore of 577 ft. The floor area of each of the eighteen floors in the main section will be more than 200,000 sq.ft., and there will be a five-story tower in addition. The cubical contents of the Merchandise Mart will be 53,000,000 cu.ft., as compared with 25,370,000 cu.ft. in the Chicago Furniture Mart and 24,000,000 cu.ft. in the Equitable Building in New York City, its nearest rivals in size. The estimated cost is $30,000,000. Construction work will start in the near future. The building was designed by Graham, Anderson, Probst & White, architects.

Syracuse Assumes Operation of Garbage-Reduction Works

Operation of the garbage-reduction works owned by the city of Syracuse, N. Y., but run under a five-year contract by the Cobwell Reduction Company, was resumed by the city on the expiration of the lease, May 1. The city authorities recently took bids for two or more methods of garbage disposal, but failed to decide what plan to adopt. The company named above was a high bidder. The plant has been put under the operating direction of William Gyatt, of the engineering staff of the intercepting sewer board.

Engineering School at University of Southern California

Twenty-three years after the inauguration of the first courses in engineering at the University of Southern California a college of engineering has been created at that institution. The demand for engineering education in the Southwest and the need of industries in southern California for centralized and adequately equipped engineering laboratories has led the board of trustees of the university to authorize this college. Philip S. Biegler, professor of electrical engineering for the past five years, has been appointed acting dean by President Rufus B. von KleinSmid. The new college, which will open in September, will be made up of five major divisions with separate chairmen as follows: chemical engineering, Prof. Wilfred W. Scott; civil engineering, Prof. Robert M. Fox; electrical engineering, Prof. Philip S. Biegler; mechanical engineering, Prof. Thomas T. Eyre; petroleum engineering, Prof. Allan E. Sedgwick.

In addition to the four-year courses leading to degrees of bachelor of science in any of the divisions of engineering and master's degrees for graduate work, there has been established the degree of civil engineer for distinction in the practice of engineering.

Professor Biegler, who will head the new college, came to Southern California five years ago after thirteen years of teaching at the universities of Iowa, Illinois and Purdue and at Washington State College. He also spent a number of years with the Commonwealth Edison Company in Chicago and the Washington Water Power Company in Spokane in an engineering capacity. From 1918 to 1921 he was associate engineering editor of *Electrical World*. He is a graduate of the University of Wisconsin.

Lachine Canal Tunnel Approved

Plans for a $2,500,000 tunnel under the Lachine Canal at Wellington St., Montreal, which will link up the outlying municipalities of Verdun, Lasalle and St. Gabriel with the heart of Montreal, have been approved by the executive committee of the Montreal city council. The proposed tunnel will be about 1,000 ft. long and will consist of four tubes, each 20 ft. in diameter. Two of the tubes will be used by the street cars and the other two by vehicular traffic, a wide sidewalk being provided in one tube for pedestrians. The federal government, city of Montreal and the Montreal Tramway Company will each pay one-third of the cost.

Permit Sought for Power Project on Clarks Fork, Columbia River

Hugh L. Cooper has applied again for a preliminary permit covering a project in Z Canyon on Clarks Fork of the Columbia River near the Canadian boundary. The Federal Power Commission in April of last year rejected an application for a preliminary permit covering this project because of the improbability of obtaining state rights and because of the lack of potential market. The objection of the state, which was based upon possible interference of this project with development of the Columbia Basin, seems to have been removed and Colonel Cooper advises that he now is in a position to demonstrate his ability to market the power. Some 200,000 hp. of primary power is available at the site, and it is planned eventually to install 350,000 hp. Favorable reports have been submitted by the Forest Service and the Bureau of Reclamation with the usual provisos for the protection of the interests under their charge. It is understood that a favorable report on the project has been submitted to the War Department.

Canada Research Council to Expand Its Work

As an initial step to the proposed expansion of the work of the National Research Council of Canada, Henry Marshall Tory, president of the University of Alberta, has resigned that position and accepted the appointment of president of the National Research Council. The new laboratories to be erected in Ottawa this season at a cost of $750,000 will combine the functions of the Bureau of Standards at Washington and the Mellon Institute at Pittsburgh. When these laboratories have been established, it will be possible for the council to carry on national standardization and research service similar to that undertaken in other important industrial countries. The program contemplates the expenditure of $3,000,000 over a period of ten years.

Latest Action on Wanaque Aqueduct and Aerator Project

The North Jersey District Water Supply Commission on May 3 voted to carry out the general recommendations of George W. Fuller, consulting engineer, New York City, to utilize the reservoir head for the aqueduct, install pumps to lift the water from the aerator to the aqueduct when the aerator is in use and reinforce the short length of tunnel at the dam and also the tunnel through the Watchung Mountain to carry the extra pressure. This is in line with action already reported but objected to later by the city of Newark (*Engineering News-Record*, April 26, p. 678, and May 3, p. 711), but with an understanding that the extra construction cost involved will be met by the portions of the district for whose benefit the changes in the plans and specifications already before the eight cities and towns were made, more particularly by Paterson, Passaic and Clifton just below the first take-off, and Kearny, at the lower end of the aqueduct. The plans and contracts now before the several communities will be recalled and another set of supplementary contracts submitted, including a new estimate of cost beyond the expenditures originally authorized.

PRESIDENT COOLIDGE GREETS AM. SOC. C. E. IN WASHINGTON APRIL 25

Washington Notes

President Asks Changes in Flood Bill and Favors Boulder Canyon Measure—Highway Legislation

By Paul Wooton
Washington Correspondent

A WHITE HOUSE announcement May 4 made it clear that the President favors the passage of the Swing-Johnson bill. In his indorsement the President is understood to have placed emphasis on the flood-control function of the dam. He referred to power generation as "incidental." He feels that Congress should have confidence in the mature consideration of the Interior Department and accept its recommendations. There is some doubt as to the ability of its advocates to bring the bill to a vote in the face of a determined filibuster led by the Arizona Senators. In the House the measure has been assigned a place well down on the legislative program.

Muscle Shoals

The Muscle Shoals bill also has been given a place on the tentative program, but a strong fight is being made on the bill recommended by the military affairs committee on the ground that it is unfair to private industry. The bill sets up a government corporation, turns over to it, without interest, rental or any other charge, properties which cost $140,000,000, gives it $10,000,000 for operating capital and allows it to keep the revenue accruing from the sale of power.

Highway Legislation

An agreement has been reached whereby the highway bill, authorizing appropriations for an additional two years, will be called up in the House under a suspension of the rules. This is expected to prevent an effort to amend the bill increasing the rate of federal aid from $75,000,000 to $100,000,000 annually. The House has passed a separate bill authorizing an additional $3,500,000 for roads on unappropriated public lands. The measure already has passed the Senate.

Flood Control

Changes made in the flood-control bill seem to have insured for the measure a place on the statute books. After the conferees had agreed and the conference report actually had been presented in the House, the President called the conferees to the White House. He specified the further changes that would be necessary to secure his approval of the bill. These changes were promptly approved by the conferees and all indications are that both the Senate and the House will approve the conference report.

In the section of the bill in which the United States is required to purchase flowage rights, the President insisted on the insertion of the word "additional" so as to make the first sentence of the section read: "The United States shall provide flowage rights for additional destructive flood waters that will pass by reason of diversions from the main channel of the Mississippi River; provided that in all cases where the execution of the flood-control plan herein adopted results in benefits to property such benefits shall be taken into consideration by way of reducing the amount of compensation to be paid." By the insertion of the one word it is believed that the government will have to make no payments in the backwater areas on the lower stretches of the Red, the Yazoo, the Arkansas, the St. Francis and along the Atchafalaya. Some are of the opinion that the insertion of the word will mean that no payment will have to be made for lands overflowed in 1927.

The President also required an important change in section 3 of the bill, under which damages to riparian owners were to be paid, whereby such payments are limited to lands "which are not now overflowed or damaged."

Quebec Authorizes Power Project on St. Lawrence

The Province of Quebec has authorized the Beauharnois Light, Heat & Power Company to divert 40,000 sec.-ft. from the St. Lawrence River between Lake St. Francis and Lake St. Louis for power development. The company has agreed to develop 100,000 hp. within five years and 500,000 hp. within ten years, and will deposit $500,000 as a guarantee that it will carry out its undertaking. Excess power not needed in Quebec may be exported to Ontario, but not to the United States. The estimated cost of the project is $30,000,000.

Wachusett-Coldbrook Tunnel Contracts Signed

Contracts for building the Wachusett-Coldbrook water tunnel were let on May 2 by the Metropolitan District Water Supply Commission of Boston, Mass. Johnston P. Porter, R. Frederick Porter and Lawrence T. Porter received the contract for the easterly portion, and the same men, together with James J. Coughlan & Sons, Inc., received the contract for the westerly part. A. C. Dennis, a well-known tunnel engineer, will have general charge of the entire work for the contractors. The tunnel, some 14 miles in length, will divert the flood flows of the Ware River to the existing Wachusett reservoir of the Metropolitan water-works system. Eventually it will be extended to a storage reservoir on the Swift River, making a total length of some 25 miles. Excess flood waters from the Ware will flow westward to storage, then eastward through the whole length of the tunnel. Frank E. Winsor is chief engineer of the commission. The estimated cost of the work under contract is $9,000,000. Already six shafts have been sunk and headings started from them, under a contract held by the Coughlans.

Steel Derrick Plunges 24 Stories Into Crowded New York Street

Crashing into crowded 41st St., New York City, from a height of 24 stories, a steel guy erection derrick spared pedestrians and vehicles alike as it struck the pavement and crumpled against the cornice of a building across the street from the one from which it fell. The derrick, with a 90-ft. boom and 80-ft. mast of lattice section, was being used in the erection of the 56-story Chanin Building, Lexington Ave. and 41st and 42nd Sts., and was set up on the 24th floor level one bay back from the building line. Striking a fascia girder at the nineteenth floor level, the falling derrick was deflected across the street, wrecking a small section of the roof and coping of an old four-story brick building. This deflection probably saved the lives of a score of bricklayers working on a scaffold at the seventh floor level of the new building. At the time of collapse a 12-ton column section was being hoisted from a delivery truck. The truck was demolished by the falling column.

As yet the cause of the collapse remains undetermined. The foot block remained in its place on two 14-in. 171-lb. H-column sections used as beams over a 16-ft. span. All the lower ends of the guy wires were left sound, the guys breaking away at the mast spider. Eyewitnesses claim the boom and mast fell as a unit, indicating the failure of a guy line rather than a structural collapse. The derrick was rated 20 tons capacity.

The accident occurred about 2:30 p.m., May 3, just at the end of the noon rush hour.

The Chanin Construction Company is the general contractor on the building. The derrick was owned by Levering & Garrigues, who have the contract for steel erection.

Serrill Heads A.E.S. Committee

The American Engineering Standards Committee has announced the election of William J. Serrill, assistant general manager of the United Gas Improvement Company, of Philadelphia, as chairman. Mr. Serrill has been identified with the national standardization movement for several years as a representative of the American Gas Association. Cloyd M. Chapman, New York, will be vice-chairman.

St. Mary's River to Be Dredged

Specifications have been completed and money appropriated for the dredging of 10 miles of the Middle Neebish channel of the St. Mary's River near Sault Ste. Marie, Mich. The channel will be 500 ft. wide and 22 ft. deep. Eight dredges will work for sixteen hours a day for three seasons to complete the work, which will cost approximately $3,842,000. Work will probably start in August or September.

Engineering Societies

Calendar

Annual Meetings

AMERICAN ROAD BUILDERS' ASSOCIATION, Washington, D. C.; Annual Meeting, Washington, D. C., May 11.

AMERICAN ASSOCIATION OF ENGINEERS, Chicago; Annual Convention, El Paso, Tex., June 4, 5 and 6.

AMERICAN WATER WORKS ASSOCIATION, New York; Annual Convention, San Francisco, week of June 11.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION; Annual Meeting, University of North Carolina, June 25.

AMERICAN INSTITUTE OF QUANTITY SURVEYORS, Chicago; Annual Convention, New York, June 25-27.

AMERICAN SOCIETY FOR TESTING MATERIALS, Philadelphia; Annual Meeting, Atlantic City, June 25-29.

AMERICAN SOCIETY OF CIVIL ENGINEERS; Annual Convention, Buffalo, N. Y., July 18-20.

THE CONFERENCE OF STATE SANITARY ENGINEERS will hold its next meeting in Chicago on Oct. 13-15, with headquarters at the Hotel Stevens.

AMERICAN SOCIETY OF CIVIL ENGINEERS—The St. Louis Section listened on May 4 to a talk by W. W. De Berard, associate editor, *Engineering News-Record*, on "The Economics of Flood Control." Mr. De Berard also addressed the Press Club of Chicago on April 28 on the same subject.

THE NEW YORK CHAPTER of the American Association of Engineers held its annual dinner on May 3, with J. H. Griffin, president of the chapter, as toastmaster. The speakers were Edmund J. Fitzmaurice, national director of A.A.E.; D. B. Steinman, past national president of A.A.E., and Arthur Sheridan, president New York State Society of Professional Engineers and Land Surveyors. The talks pointed out the need for the individual engineer to seek public recognition of his work in order that the entire engineering profession receive its due recognition. The licensing of all engineers and more cooperation within the ranks will do much to raise the standing of the profession, the speakers maintained. J. A. L. Waddell also spoke briefly.

Personal Notes

GEORGE R. WADSWORTH, of New York City, has been appointed assistant commissioner of correction of the State of New York, and will be in charge of prison industries and production. During the past four years Mr. Wadsworth has served as director of the bureau of planning and operating research in the Division of Architecture, State Department of Public Works, in which position he made fundamental determinations which served as a groundwork for the large institutional planning program now under way. Mr. Wadsworth was graduated from the Massachusetts Institute of Technology in 1898, and one of his earliest undertakings was the designing and construction of the New York Central Railroad passenger yard and station at Albany. He also served as resident engineer during the Grand Central Terminal improvements in New York City. From 1909 to 1912 he was consulting engineer of the Metropolitan Improvement Commission of Massachusetts.

A. C. DENNIS, formerly engineer of construction on the Canadian Pacific Railway and subsequently superintendent for the contractors in the construction of the Rogers Pass tunnel for that railway, is to have charge of the construction of the 14-mile Wachusett-Coldbrook tunnel for the Boston water-supply system as the representative of the contractors.

GEORGE S. BARTLETT, of the Universal Portland Cement Company, Chicago, having attained his 70th birthday on April 2, sailed on April 28 with Mrs. Bartlett for Montevideo and Buenos Aires. After spending several weeks in Uruguay and in the Argentine they will sail from Buenos Aires to Genoa, Italy, and will motor from Genoa to one of the Channel ports and thence go to England. They expect to return to the United States about Aug. 1.

WALDEMAR KAEMPFFERT, science and engineering editor of the New York *Times*, has been appointed director of the Chicago Industrial Museum, which will occupy the Fine Arts Building of the 1893 World's Fair. The museum, made possible by a gift of $3,000,000 by Julius Rosenwald and $5,000,000 given by the South Park Commissioners for reconstruction of the building, will house a well-planned display of engineering and industrial exhibits.

FRANCIS C. SHENEHON announces that he has resumed general engineering practice in Minneapolis, specializing in water power, waterways, canals, docks, terminals, flood control and reservoirs. From 1909 to 1917 Mr. Shenehon was dean of the college of engineering of the University of Minnesota and since then has been consulting engineer for the Sanitary District of Chicago and for various public utility corporations.

JAMES M. KIER, for the past twelve years associated with the Phoenix Utility Company as construction superintendent, has resigned. He has associated himself with George M. Allen, formerly traffic manager of the Phoenix Utility Company, in a general contracting business, with offices located at Johnson City, Tenn.

EDWARD P. LUPFER, consulting engineer and builder of the Peace bridge between the United States and Canada, has been elected president of the Buffalo Section of the American Society of Civil Engineers, succeeding Lynn L. Davis. Mr. Lupfer has been vice-president during the past year. THERON M. RIPLEY, chief engineer of the greater motorway system of Erie County, was elected vice-president, and GEORGE F. PRONG was re-elected secretary-treasurer.

ERNEST E. LOTHROP has accepted a position as general superintendent for P. J. Shea on water-works construction at Delmar, in Bethlehem Township, N. Y.

C. T. FELIX has recently been appointed resident engineer with the Alabama State Highway Department. He is at present located in Berry, Ala. Mr. Felix was formerly with the North Carolina State Highway Commission and the Florida State Road Department as resident engineer, and with M. C. Winterburn, Inc., of Jacksonville, Fla., as construction engineer.

WINSTON ATTEBERRY, who for the last two years has been employed in Nicaragua by the Bragmans' Bluff Lumber Company, of New Orleans, first as terminal engineer, then as resident engineer on railroad construction and finally as engineer on location and exploration to the interior, has returned to the United States for a vacation. His present address is Eunice, La.

CHARLES G. MASSA has withdrawn from the engineering firm of Tribus & Massa, of New York City, but the business will be continued under the same firm name with Louis L. Tribus at its head and C. G. Massa, J. T. Fetherston, C. R. Bettes and Q. F. Disher as special associates. The firm specializes in consultation, design, supervision, management, appraisal, water, sewage, refuse, power, roads and town planning.

C. W. STURTEVANT has returned to New York City from Xicotencatl, Mexico, where he has been engaged as superintendent of construction of a reinforced-concrete diversion dam and its distributing canals for the Rio Monte irrigation project, built for the Mexican government by the J. G. White Engineering Corporation.

T. A. BEDFORD, California state highway division engineer, with headquarters at Eureka, has resigned to join the Kaiser Construction Company, of Oakland, Calif., in a large highway building program in Cuba.

Obituary

JOHN LALLY, SR., president of the Lally Column Company, Cambridge, Mass., died on April 12, 1928.

Sir EBENEZER HOWARD, founder of the garden city movement in England, died on April 30 at the age of 78. Since he visualized delightful towns with tree-lined roads and open spaces 30 years ago, he has seen the cities of Letchworth and Welwyn, among others, arise in England. Recently the idea has spread to this country, and like communities have been built on Long Island and another is planned in New Jersey.

Construction Equipment and Materials

A Section Devoted to What the Manufacturer Is Doing for the Engineer and Contractor

Novel Exhibit Maintained by Cleveland Engineers

More space is being added to the headquarters of the Cleveland Engineering Society in order to give its permanent exhibit of engineering equipment and materials an opportunity to grow. The exhibit was started more than a year ago and it has proved so attractive to the manufacturers that an expansion program is necessary.

The accompanying illustrations show

how the exhibits are arranged around the sides of the large meeting room, used not only by the Cleveland Engineering Society but by such organizations as the sections of the American Society of Civil Engineers, American Institute of Mining and Metallurgical Engineers, American Chemical Society, American Society of Heating and Ventilating En-

gineers, American Society of Mechanical Engineers, American Society for Steel Treating, American Welding Society and the Society of Automotive Engineers. The alterations now under way will increase the capacity of the room to accommodate more than 300 persons and provide about 1,800 sq.ft. of display space.

Research Laboratory for Sand and Gravel

A research laboratory for the sand and gravel industry was inaugurated May 1 under the direction of Stanton Walker, director, and C. E. Proudley, assistant director of the engineering and research division of the National Sand and Gravel Association. The laboratory is located in the basement of the Munsey Building in Washington, D. C., in which building the headquarters of the association are also maintained.

The policies of the laboratory will be controlled by the association's committee on engineering problems. The laboratory was formed by the board of directors of the association because it felt that before educating the consumer, the association should equip itself with full knowledge concerning the products of the industry which it represents. The committee on engineering problems of the association consists of F. D. Coppock, chairman; Alex. W. Dann, John Prince, Hugh Haddow, Jr., H. V. Owens and R. J. Potts.

Bolt, Nut and Rivet Industry Plans Distribution Conference

A plan for a national conference to discuss the question of proper sales of products which are distributed either all or in part through hardware jobbing and mill supply channels is being sponsored by the Bolt, Nut and Rivet Manufacturers' Association, Pittsburgh, Pa., Charles J. Graham, president. The bolt, nut and rivet industry has been using a plan since 1927 which it believes has proved satisfactory both to manufacturers and to distributors. The basic principle of the plan is the elimination of the manufacturer as the distributors' leading competitor and the changing of the attitude of the distributor from one of antagonism to one of co-operation in a proper and lawful effort to stabilize market conditions. The association believes that its plan is applicable to all competitive lines, and it is hoped that the planned national conference will include as many of the different industries as possible.

In developing the plan for the bolt, nut and rivet industry, committees have been formed in practically every important jobbing center in the country. The chairmen of these committees will be invited to attend the conference and it is suggested that each industry appoint five executives to represent the industry as a whole. If a well-represented meeting can be held, the sponsors feel that a positive step will have been made toward a desired stabilization of industry.

Business Notes

COX & SILVER, INC., Birmingham, Ala., has been incorporated by Oliver Cox and Ralph Silver, taking over, May 1, the business of the Cox Advertising Agency, which has been operated by Mr. Cox for the past ten years. Mr. Silver, who for the past twelve years had been advertising manager of the American Cast Iron Pipe Company, joined Mr. Cox last December with the purpose of incorporating the business as soon as the details could be worked out.

CEMENT GUN COMPANY, INC., Allentown, Pa., announces that a change has recently been made in its contracting and sales department in Chicago, and that it is now represented by A. C. O'Donnell and R. N. Gillespie.

WESTINGHOUSE ELECTRIC & MFG. COMPANY has furnished the Shell Pipe Line Company with fourteen electrically driven centrifugal pumps for installation on the latter's lines between McCamey, Tex., and Cushing, Okla.

New Developments

Develop New Mechanical Process for Making Sand Cast Pipe

A new method recently developed and now used by the McWane Cast Iron Pipe Company, Birmingham, Ala., places the manufacture of sand cast pipe on a mechanical basis. By the new method a pipe claimed to have greater strength

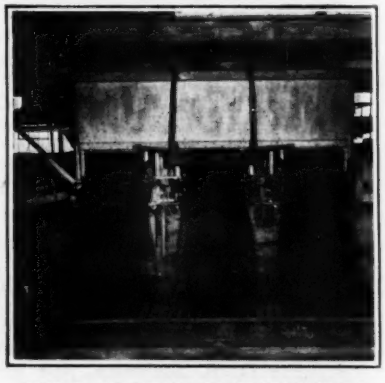


FIG. 1—PLACING SAND IN MOLDS

and lighter weight than the old-time sand cast pipe is made in stationary horizontal molds in green sand and with green sand cores. The pipe as now manufactured is made in 16 ft. lengths in accordance with the company's specifications. Sizes soon to be available will range from 4 to 12 in.

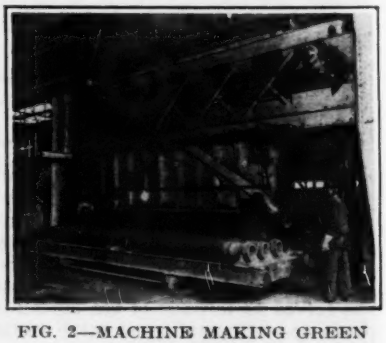


FIG. 2—MACHINE MAKING GREEN SAND CORES

Whereas in the older methods of manufacture, pipe has been cast vertically between dried molds and cores, by this new process, a patented device

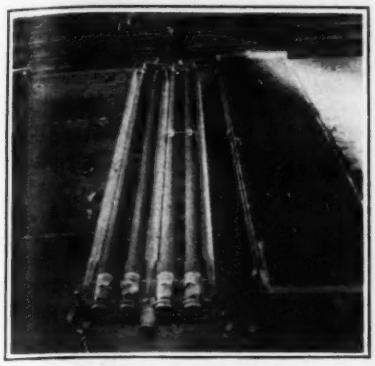


FIG. 3—SETTING GREEN SAND CORES IN GREEN SAND MOLDS

for centering the cores in the mold permits the pipe to be cast while the molds are in a horizontal position and without the necessity of drying either the molds or the cores. Also in the older method a certain number of molds were prepared and poured, constituting a day's work. In the McWane process the same flasks are used over and over again, flasks, sand and cores each moving in its own separate path to the assembling point. Sand-handling ma-

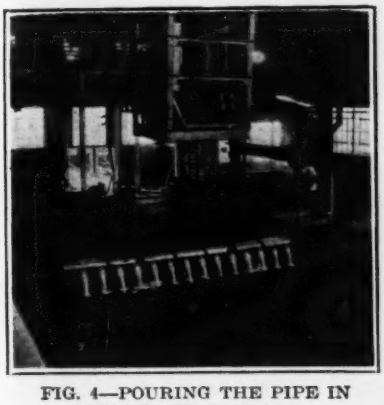


FIG. 4—POURING THE PIPE IN HORIZONTAL MOLDS

chinery, ramming, core-making shake-out machines and a battery of overhead cranes work together as one unit. As soon as the molds are ready they are taken to the casting zone. Several lengths of 16-ft. pipe are poured in one operation, followed immediately by core-bar pulling machines, the mechanical shake-out and the return of the flasks to the sand-handling equipment for immediate further use.

Large Water-Softening Plant for Boiler Feed Water

What is claimed to be the largest complete water-softening system yet built is under construction by Wm. B. Scaife & Sons Co., Oakmont, Pa., for the Carnegie Steel Co. at its Duquesne, Pa., works. An ultimate capacity of 8 m.g.d. of softened water will be used for boiler feed purposes. The system will be of the Scaife intermittent type, wherein the water will be treated and allowed time for reaction and sedimentation in four 540,000-gal. tanks. After the water is softened it will pass through eight concrete filters, each of a capacity of 1 m.g.d. Filtered water will be discharged into a clear well, from which it will be delivered by pumping to an 800,000-gal. filtered water storage tank.

A Practical Plan Filing System

The handling and filing of maps and plans is a subject of prime importance to the engineer. The cost in time and effort that these records represent is sufficient to make profitable any method that saves time in reference or adds to the longevity of the records.

The method of filing engineering records by rolling the sheets individually or in small groups and filing in pigeonholes is ineffective as regards preservation, decidedly inefficient from a reference standpoint and has been generally replaced by other systems. In one of these, which may be classified as bulk filing, the sheets are kept flat, being stored either horizontally or vertically. But in bulk filing the useful life of the records is often reduced because of the difficulty of filing sheets without folds, wrinkles and protruding edges, because of the practice of thumbing the corners of sheets and because sheet contact dulls the surface and often results in a faint transfer of lines between adjacent sheets at the bottom of the pile.

A high degree of preservation can be attained by hanging up the maps and plans. The usual method of suspension is accomplished by consolidating sheets between binders so as to form a book and suspending the book from a rack. Believing that binders are objectionable because they retard access and because they do not eliminate the destructive practice of thumbing the corners of the sheets, the Globe Wernicke Company, Cincinnati, has developed a filing system

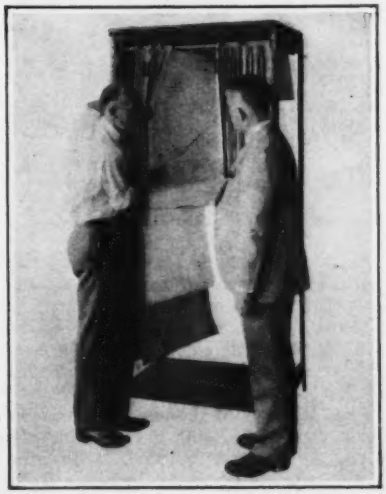


FIG. 1—INDIVIDUAL SHEET FILING BY ADHESION CLIPS

in which all accessory equipment is eliminated by the application of a loose-leaf clip to the individual sheets. The clips attach to the sheets by adhesion and may be snapped over horizontal rods as shown in Fig. 1. This method makes practical the filing of tracing, vellum and bond paper and since the clips are thicker than the sheets there can be no sheet contact. Sheets are handled individually and the clips serve as "handles" for lifting a sheet into and out of the file. On the other hand sheets may be completely exposed for reference without removal from the file. The loose-leaf clips may also be used to file sheets in groups by the use of group holder clips.

Suspended individual sheet filing lends

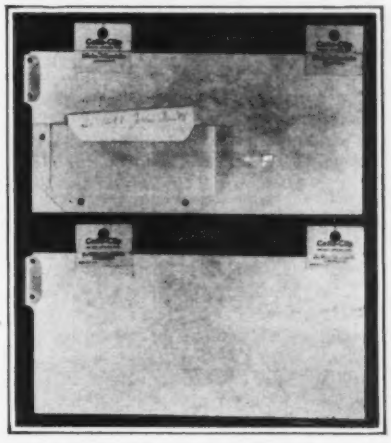


FIG. 2—METHOD OF RECORDING OUT SHEETS

itself to the application of a time saving numerical index or to an out-guide system in which guides as shown in Fig. 2 are hung in the place of the removed sheet and a notation containing the name of the possessor placed in the pocket of the out-guide.

Manufacturers and Trade Associations

Calendar

Annual Meetings

AMERICAN WATERWORKS MANUFACTURERS ASSOCIATION, New York City; annual meeting, San Francisco, Calif., June 11-14.

New Publications

Metal Lath Data—Milwaukee Corrugating Company, Milwaukee, has issued a new edition of its Milcor Manual, No. 20-E. The four kinds of Milcor metal lath are considered in detail and many practical data on reinforced concrete and expanded metal construction are included.

Business Side of Construction

Facts and Events That Affect Cost and Volume

Public Utility Group 47 Per Cent of Total April Bond Offerings

New bond offerings in April showed the largest monthly total in 1928. The outstanding increase over the preceding month was in the public utility group. This group furnished 46.7 per cent of the total April offerings of $649,965,000.

Total April offerings exceeded those of March, 1928, by 4 per cent and were only 7 per cent under the April, 1927, total. March was the only month this year which surpassed the corresponding month of 1927. Four months of this year, compared with a year ago, show a decrease of 2 per cent—1928, $2,348,180,894; 1927, $2,385,011,000. Municipal, foreign and miscellaneous issues have been offered in smaller volume than in the first four months of 1927. The totals for each group for four months of 1928 and 1927 and the percentage each group formed of the total were as follows (000 omitted):

	4 Months 1928	Per Cent of Total	4 Months 1927	Per Cent of Total
Railroad	$265,358	11.30	$239,609	10.04
Public utility	653,502	27.83	616,984	25.86
Industrial	384,301	16.36	358,150	15.01
Municipal	284,376	12.11	378,010	15.84
Foreign	561,588	23.91	596,548	25.00
Real estate	138,380	5.89	102,185	4.28
Miscellaneous	60,675	2.58	93,525	3.92
Total	$2,348,180		$2,385,011	

In April the public utility group included nine issues of $10,000,000 or more. In addition to the two listed in the accompanying table, there were several noted previously in this column—namely, $63,000,000 Associated Gas & Electric Company 4½s, $50,000,000 American Gas & Electric Company 5s, $35,000,000 Cincinnati Gas & Electric Company 4s, $25,000,000 New England Power Association 5s.

An important issue offered since May 1, 1928, was the $15,511,000 4s of Westchester County, New York. This was the usual annual refunding operation to take up notes issued during the fiscal year for improvements; and it consisted of $34,000 bridge, $70,000 hospital, $225,000 courthouse, $348,000 highway, $750,000 sewer and $14,084,000 park bonds.

This Week's Contracts

Heavy construction contracts, reported by *Engineering News-Record* in the week ended May 10, with some comparisons, total as follows:

(In Thousands of Dollars, 000 Omitted)

Week Ended	Public	Private	Total
May 10, 1928	**$42,680**	**$33,769**	**$76,449**
May 3, 1928	23,121	38,760	61,881
May 12, 1927	47,101	18,208	65,309
Jan. 1 to date:			
1928	438,159	793,694	1,231,853
1927	383,256	631,221	1,014,477

E. N.-R. Index Numbers

Cost		Volume	
May 1, 1928	**207.00***	**April, 1928**	**316***
Apr. 1, 1928	206.40	March, 1928	318
May 1, 1927	206.80	April, 1927	257
Average, 1927	206.24	Average, 1927	263
Average, 1926	208.03	Average, 1926	228
1913	100.00	1913	100

* *Correction, numbers in issue of May 3, 1928, p. 717.*

RECENT IMPORTANT BOND OFFERINGS

		Interest Rate	Yield
Municipal:			
Westchester Co., New York	$15,511,000	4	3.90
Chicago	11,655,000	4	3.75–3.95
Chicago Sanitary District	11,000,000	4½	4
Nassau Co., N. Y.	1,900,000	4	3.85
Richland, Lexington & Saluda Counties, S. C.	1,800,000	4½	4.15–4½
Aiken County, S. C., highway	1,150,000	4½	4.20
Oakland, Calif., harbor	1,000,000	4½	4
Beaumont, Tex.	837,000	4½	4.10–4½
Beaumont, Tex.	663,000	4½	4½
Schenectady, N. Y.	596,000	4	3.85
Watertown, N. Y., school	500,000	4	3.75–3.85
Public Utility:			
Cities Service Co.	$50,000,000	5	5.13
Houston Gulf Gas Co.	11,000,000	6	6.05
Houston Gulf Gas Co.	8,000,000	6½	6.60
Consolidated Water Co., of Utica, N. Y.	4,000,000	4½	4.67
New York Steam Corp. 1st	4,000,000	5	4.82
Ohio Water Service Co.	3,000,000	5	5
Columbus Electric & Power Co.	2,500,000	4½	5
Texas Cities Gas Co.	2,500,000	5½	5.63
Foreign:			
German Central Bank for Agriculture	30,000,000	6	6.62
Mortgage Bank, Chile	20,000,000	6	6.30
City of Copenhagen	12,000,000	4½	4.88
Department of Akershus, Norway	8,000,000	5	5.17
Industrial:			
Chicago Artificial Ice Co.	12,500,000	5	5½
Railroad:			
Chicago, Milwaukee & St. Paul Ry. Co.	24,000,000	4½	4.38
Union Pacific R. R. Co.	20,000,000	4	4.38

Bond elections in April included the approval of voters of financing operations of 168 municipalities which totaled $77,465,239. Taxpayers of 56 municipalities defeated bonds aggregating $96,011,900, which included $77,959,000 city of Chicago issue.

The amount of bonds approved in April was $1,687,000 greater than in April, 1927, and $1,066,000 greater than two years ago. An increase from the first of the year has been according to usual seasonal trends.

Excess of Exports Over Imports Near 1913 Level

Exports of the United States in 1927 totaled $4,864,806,000, an increase of 1.2 per cent over the 1926 figure, while imports totaled $4,184,378,000 or 5.6 per cent below the 1926 value, according to the Bureau of Foreign and Domestic Commerce. Excess of exports over imports in each of the last fifteen years was as follows:

1913	$691,421,812	1921	$1,975,883,786
1914	324,348,049	1922	719,030,636
1915	1,776,074,152	1923	375,427,117
1916	3,091,005,766	1924	981,021,266
1917	3,281,044,642	1925	683,258,248
1918	3,117,874,835	1926	377,771,869
1919	4,016,061,058	1927	680,427,591
1920	2,949,534,817		

The interchange of construction materials is shown in the accompanying tables, with exports normally in excess of imports in this group.

The most important increases in imports of construction materials occurred in structural steel and in iron and steel pipes and tubing, 33.6 per cent and 14.8 per cent, respectively.

The relation of imports to 1927 domestic production in percentages is as follows: Cement and steel bars, 1 each; iron and steel pipe, 2; asphalt, 3.7; structural shapes, 4; tile, 6.6; plate glass, 14.5; lumber, 12.6.

The relation of exports to 1927 domestic production in percentages follows: Cement, 0.4; steel bars, 2.1; iron and steel pipe, 4.6; structural shapes, 5.6; asphalt, 9; iron and steel scrap, 10.7; steel rails, 6.3.

CONSTRUCTION MATERIALS IMPORTED

Materials	(000 Omitted) 1927	1926	Per Cent Gain (+) or Loss (—) In 1927
Cement, bbl.	2,049	3,250	—36.6
Structural steel, lb.	362,542	271,265	+33.6
Asphalt, lb.	316,194	285,264	+10.8
I. & S. pipe, lb.	295,485	257,485	+14.8
Steel bars, lb.	204,954	231,781	—11.6
Plate glass, sq.ft.	16,786	26,664	—37.0
Tiles, sq.ft.	5,255	6,209	—15.4
Boards, ft. b.m.	1,745,000	1,899,000	—8.1
Laths	1,705,000	1,832,000	—6.9
Timbers, ft. b.m.	108,780	107,253	+1.4
Mahogany, ft. b.m.	72,828	68,431	+6.4
Telegraph poles	797	791	+0.8
Shingles	1,965,000	2,492,000	—21.1

CONSTRUCTION MATERIALS EXPORTED

Materials	(000 Omitted) 1927	1926	Per Cent Gain (+) or (Loss (—) In 1927
Boards, lb.	6,851,057	6,124,556	+11.9
Timbers, lb.	2,384,118	2,307,314	+3.3
I. & S. pipe, lb.	641,819	721,124	—11.0
I. & S. scrap, lb.	535,730	234,837	+128.1
Structural steel, lb.	497,779	534,877	—6.9
Steel rails, lb.	397,979	420,128	—5.3
Steel bars, lb.	307,495	375,274	—18.1
Cement, bbl.	816	974	—16.2
I. & S. wire, lb.	185,414	174,444	+6.3
Ry. track materials, lb.	47,959	62,055	—22.7
Rails, lb.	41,298	44,056	—6.3
Carwheels, axles, lb.	37,323	37,360	—0.1
Copper wire, cable, lb.	33,001	31,594	+4.5
Excavators	*411	*268	+53.4
Piling, lin.ft.	4,686	4,472	+4.8
Railroad ties	3,550	3,761	—5.6
I. & S. plates, lb.	1,904,190	2,023,719	—5.9
Logs, lb.	1,013,172	760,574	+33.2
Asphalt, lb.	770,932	293,178	+163.0

* 000 not omitted.